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THE STRUCTURAL ENGINEER

THE JOURNAL OF THE
INSTITUTION OF STRUCTURAL ENGINEERS

The President, 1960-61

Structural Considerations of a New Animal Feed Mill in Belfast, Northern Ireland
By Edward V. Finn, (Member) and Kenneth F. Shadbolt (Associate-Member)

A New Method for Predicting the Natural Frequency of Foundation-Soil Systems
by H. A. Balakrishna Rao and C. N. Nagaraj

The Design and Construction of the New Assembly Building for the Ford Motor Company Limited, Dagenham
Discussion on the Paper by D. Lax (Member) and F. T. Bunclark (Member)

Folded Plate Construction : An Investigation of Collapse Conditions
Discussion on the Paper by A. R. Dykes (Associate-Member)

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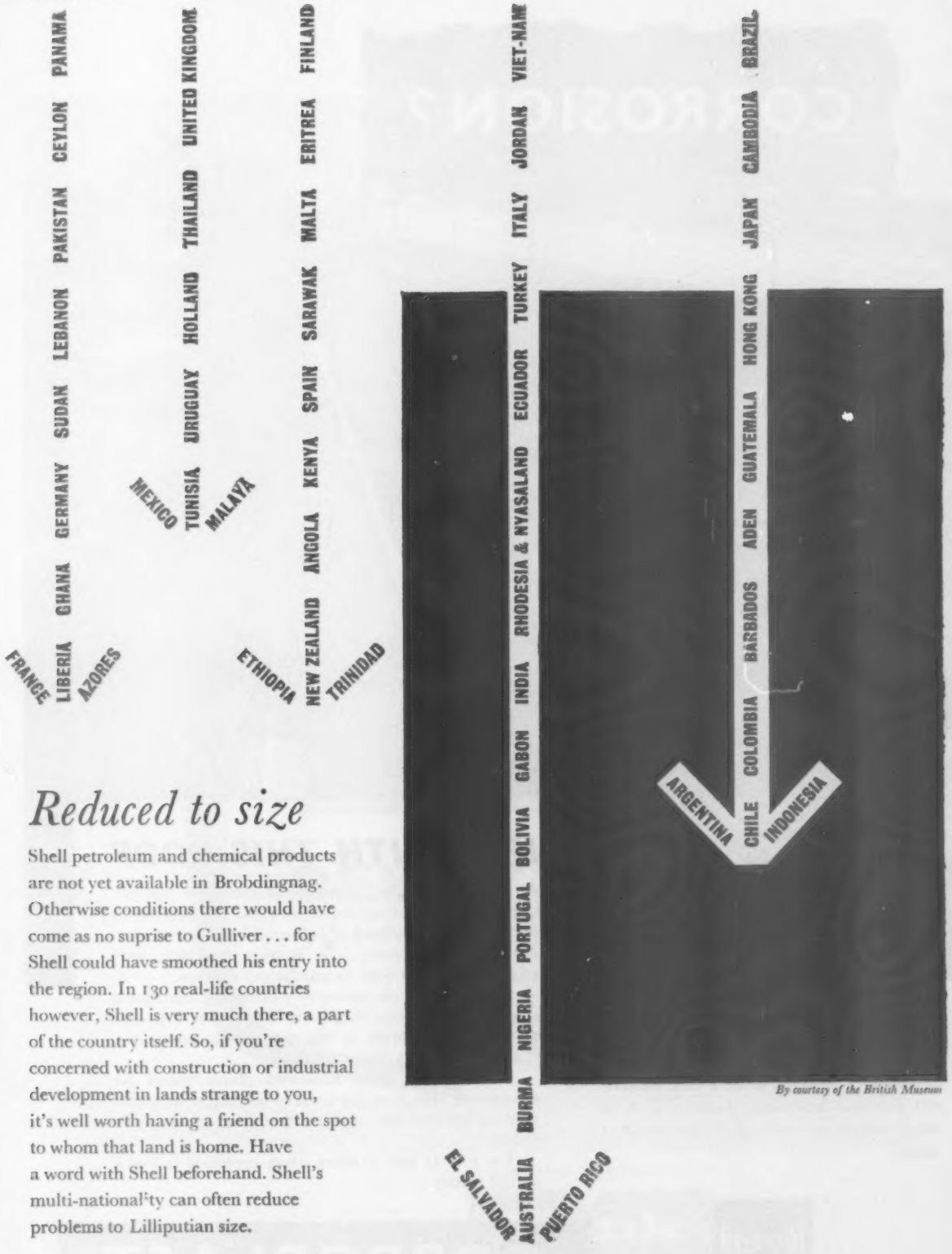

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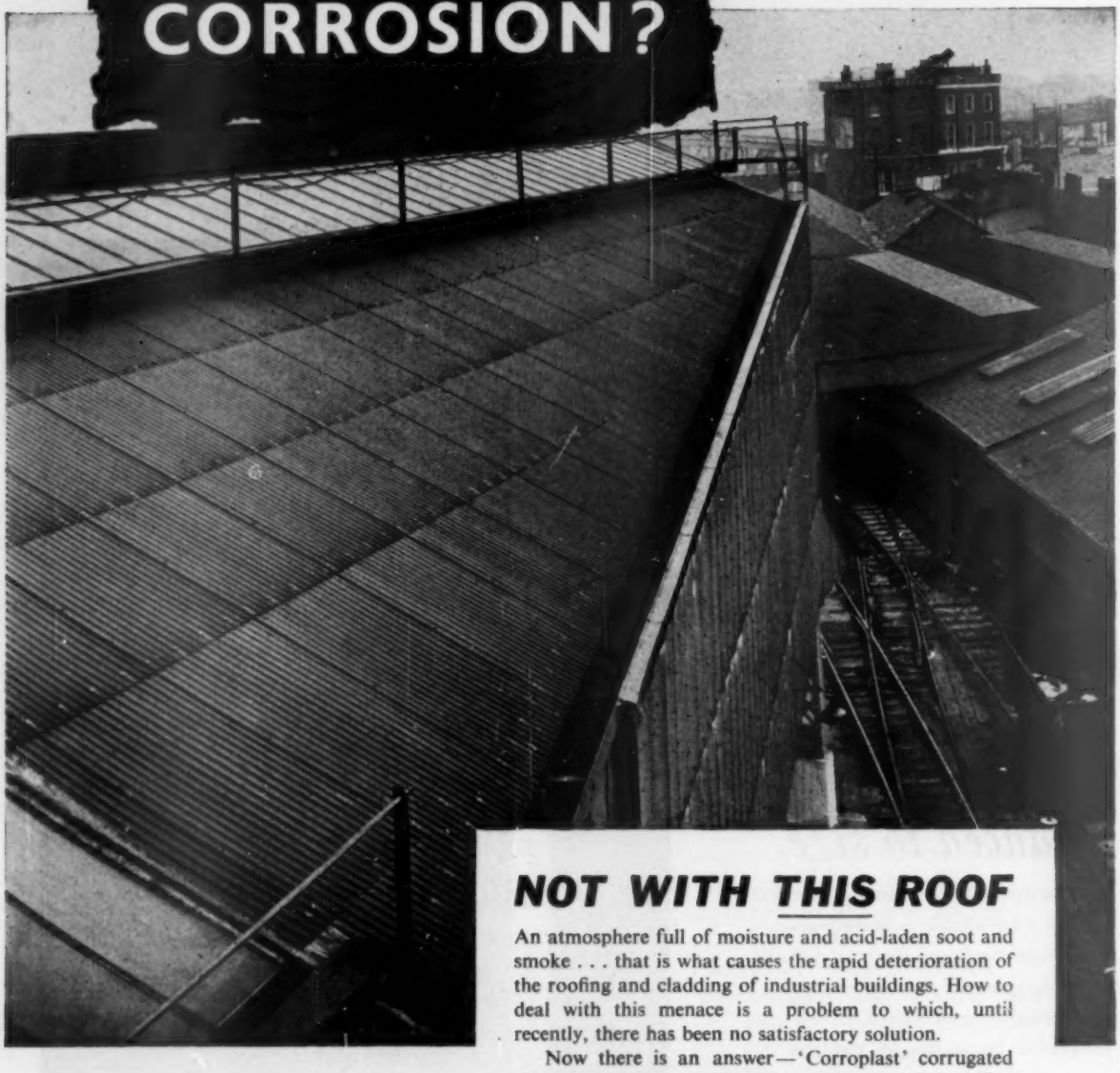

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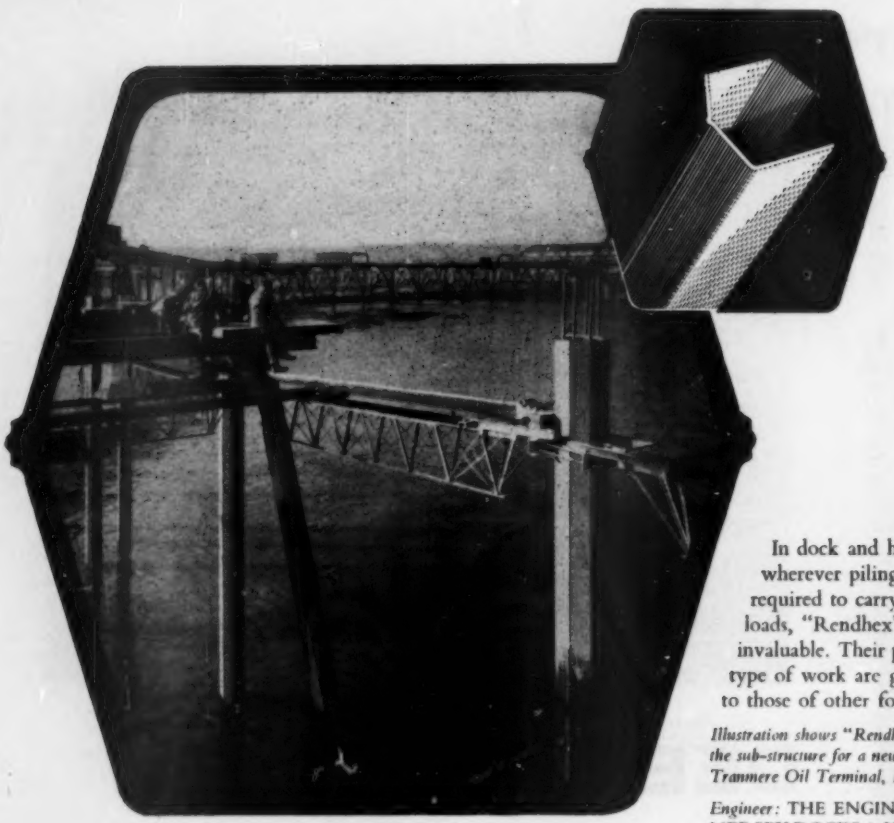

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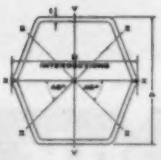

TABLE OF PROPERTIES

"Rendhex" Column No.	Size (Ins.)			Weight of Rolled Section lbs. per Lin. ft.	Weight of complete Col. (incl. Weld) lbs. per Lin. ft.	Area of complete Col. (excl. weld) Sq. ins.	Moment of Inertia (Ins.⁴)			Radii of Gyration (Ins.)			Section Moduli (Ins.³)		
	A	B	t				About XX	About YY	About ZZ	About XX	About YY	About ZZ	About XX	About YY	About ZZ
No. 3	13	12½	½	36·15	72.50	21·26	469·3	381·3	425·3	4·69	4·23	4·47	72.20	58·70	57·30
No. 4	16	16¼	½	58·30	117·75	34·30	1214	1038	1126	5·95	5·50	5·73	151·75	125·82	116·15
No. 6	20	20	0·7	80·70	162·80	47·47	2657·7	2203·5	2430·6	7·49	6·81	7·16	265·80	218·60	207·20

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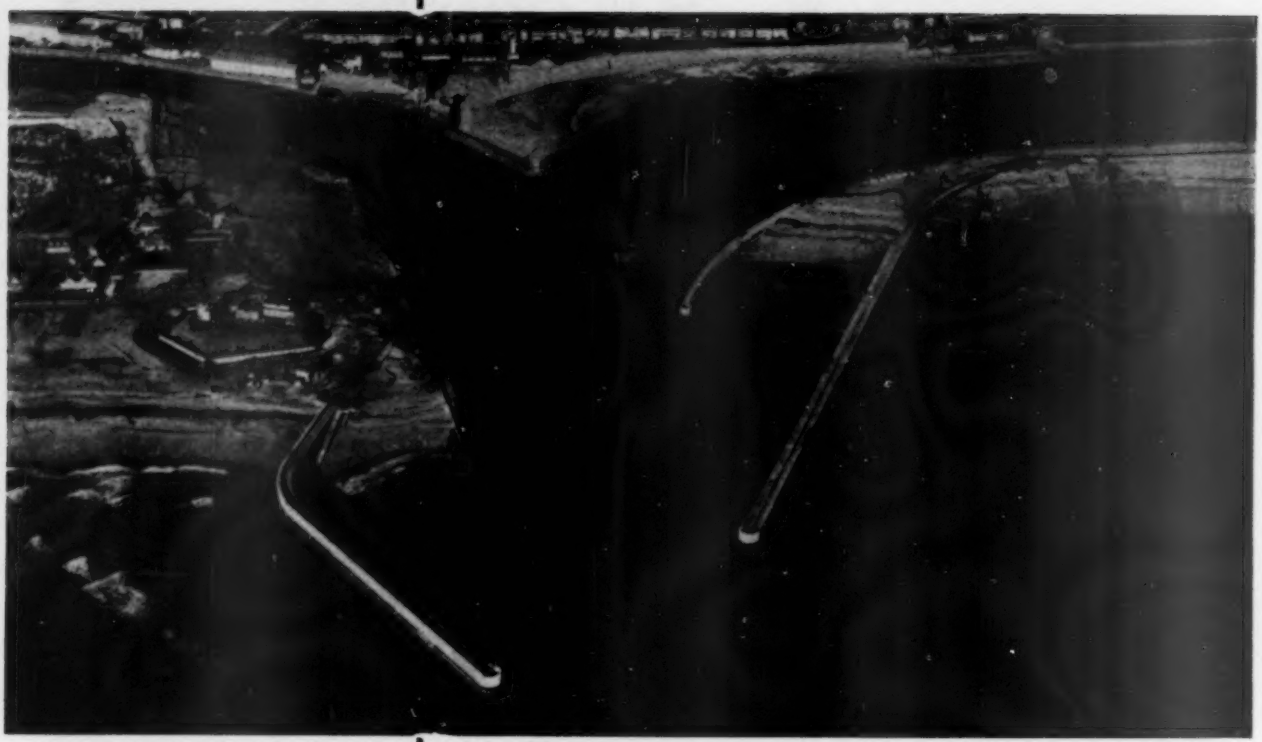

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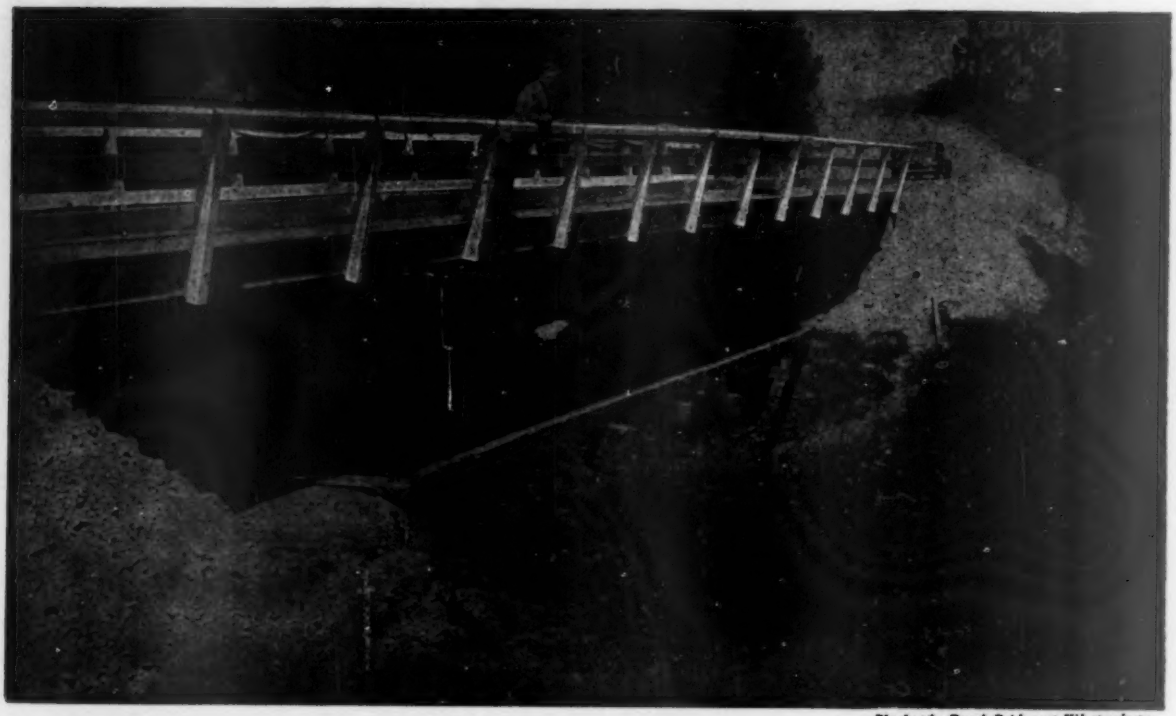

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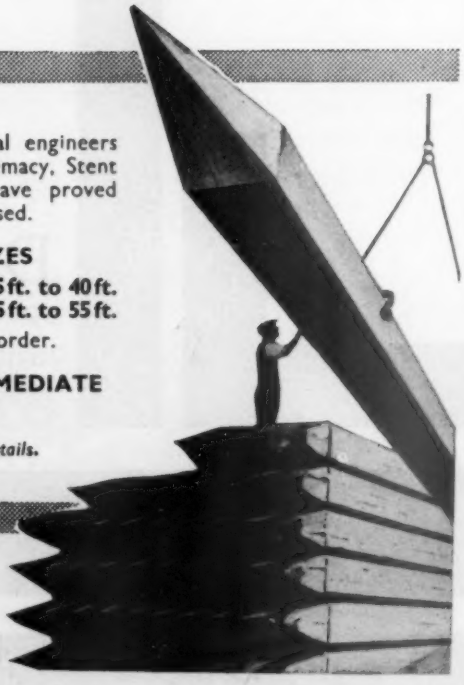

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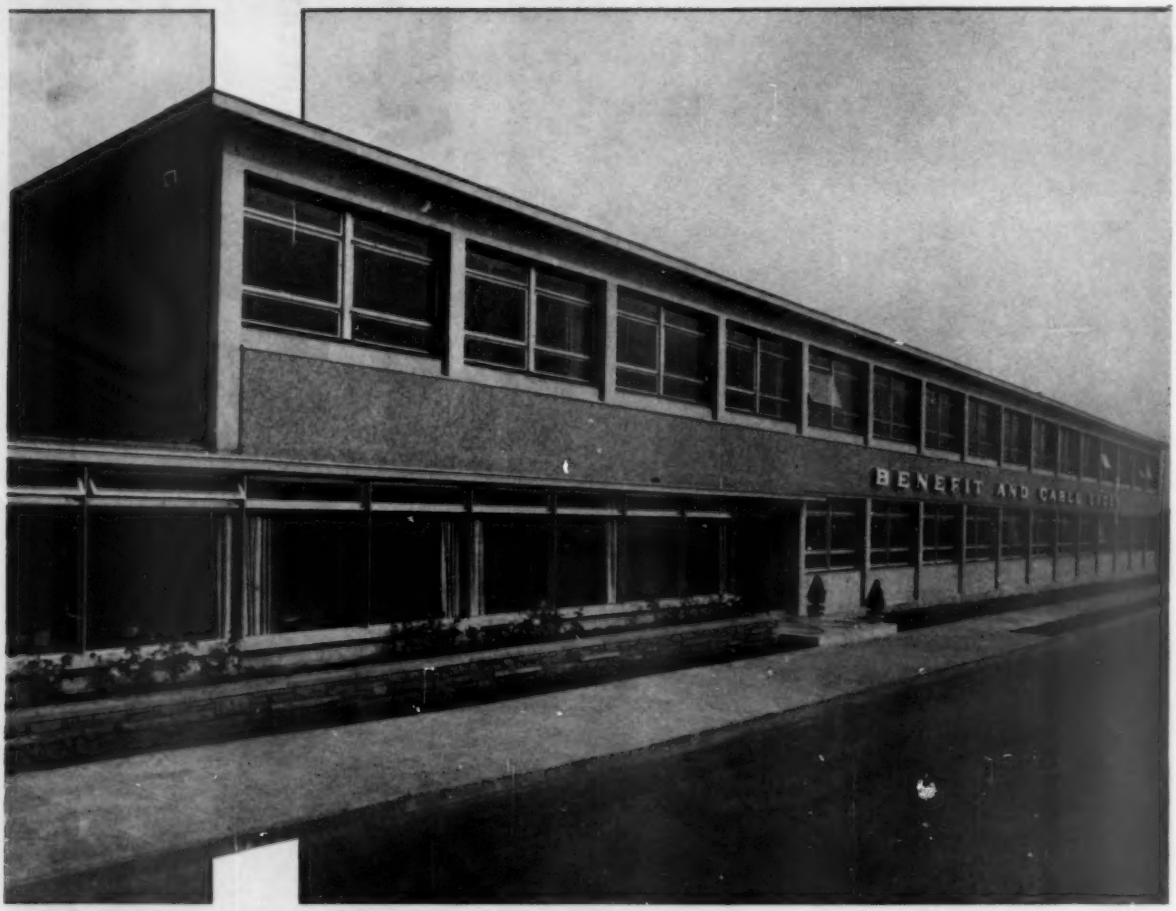

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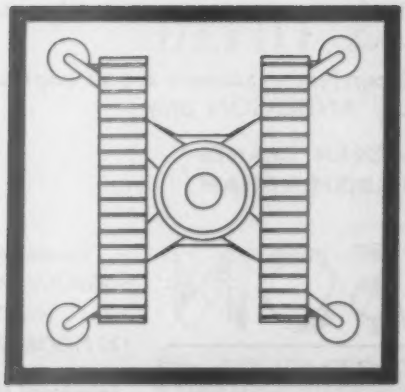

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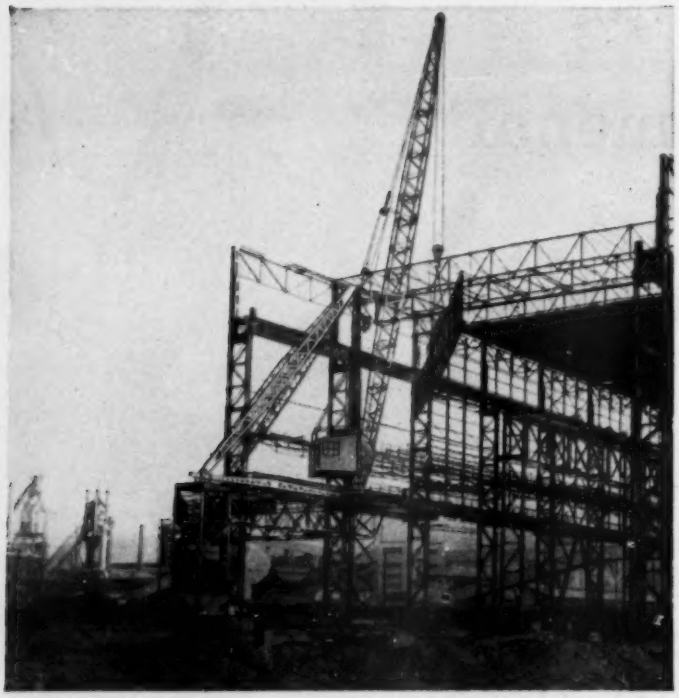

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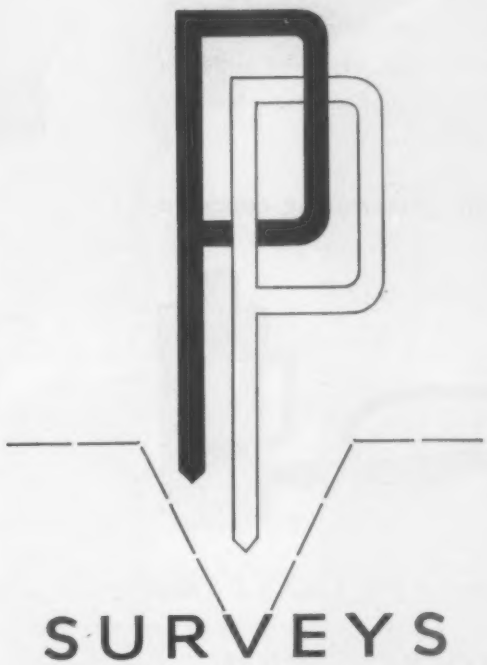

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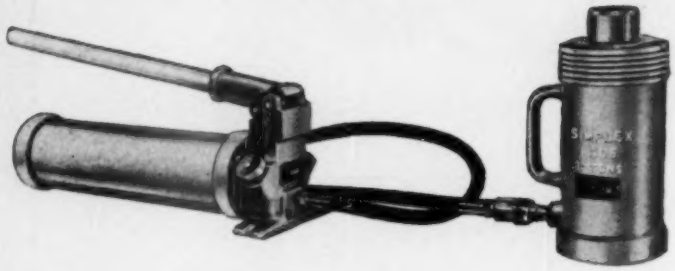

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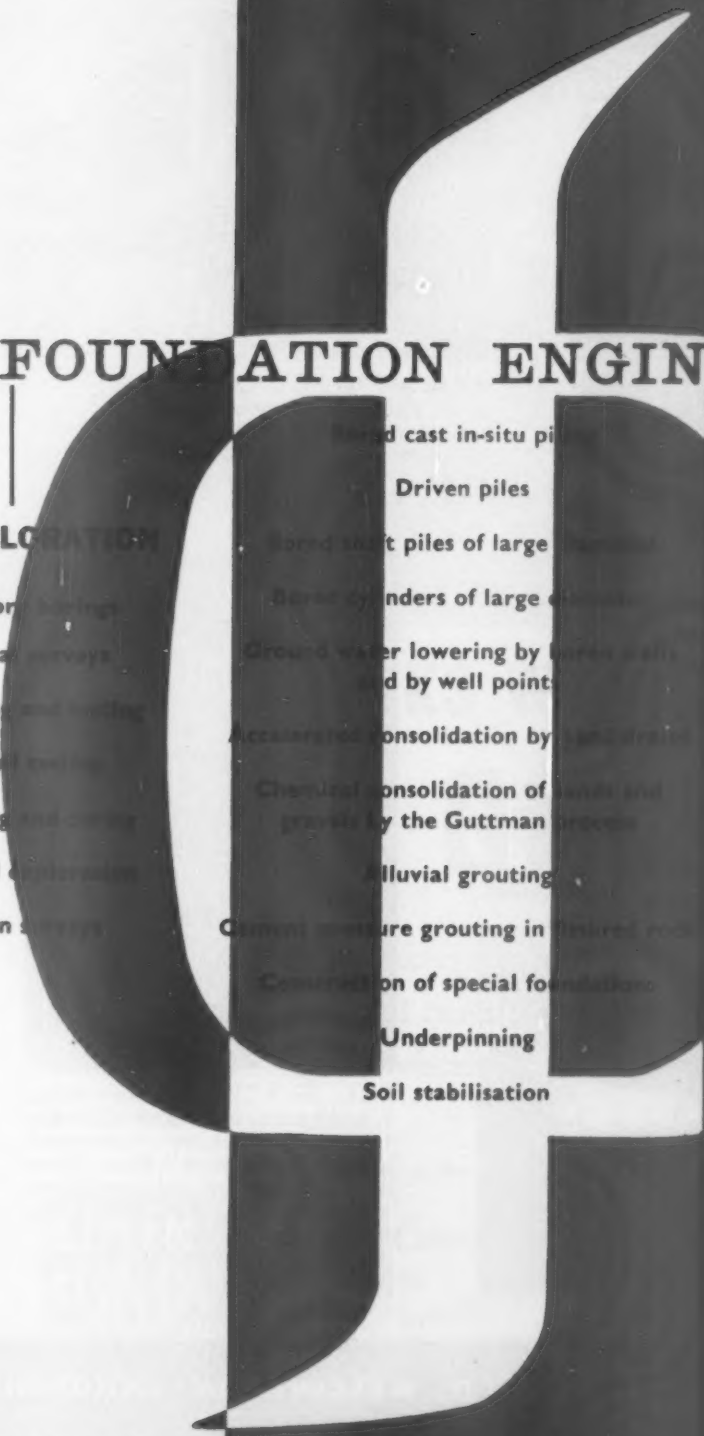

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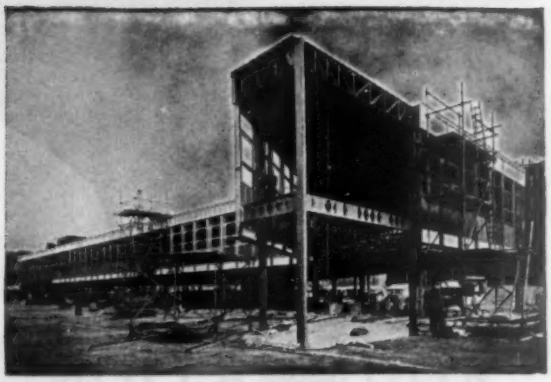

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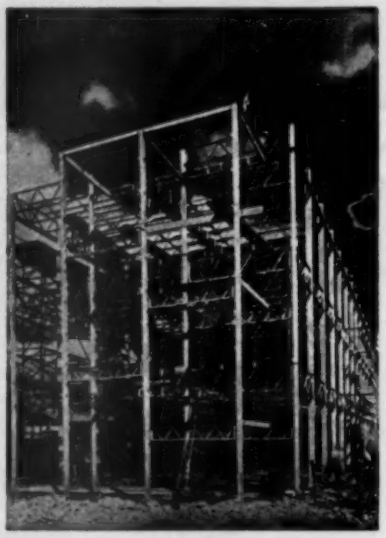

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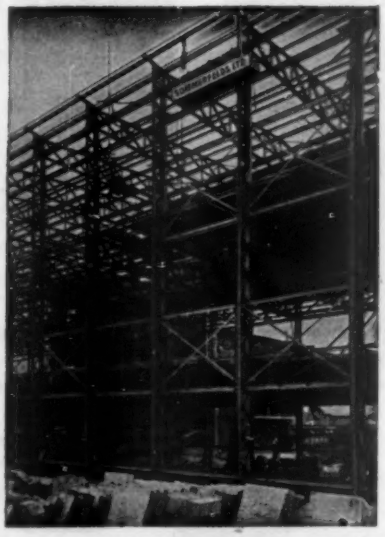

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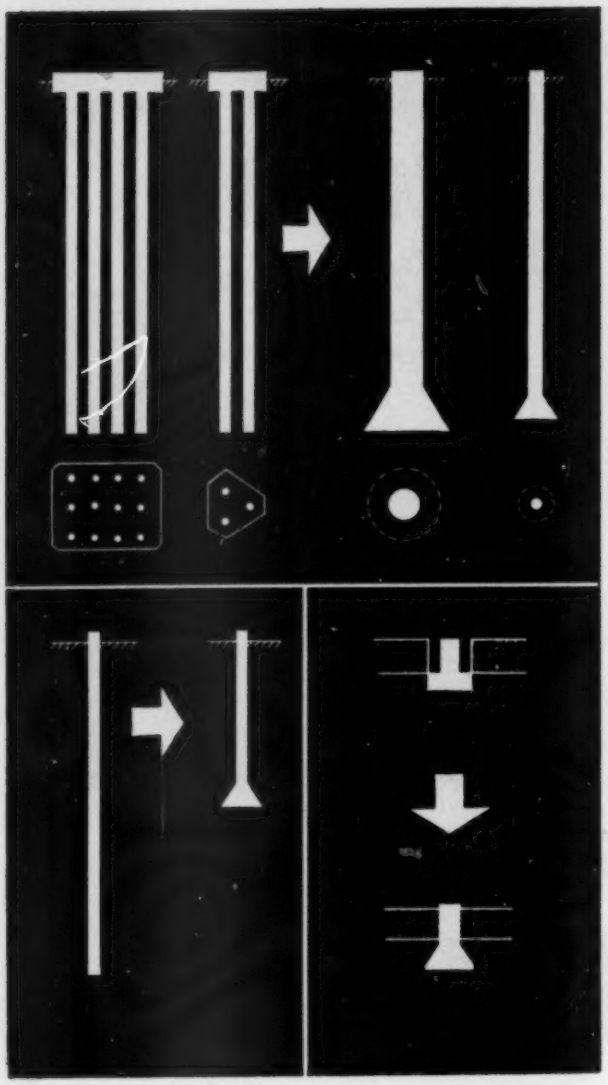

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THE STRUCTURAL ENGINEER

THE JOURNAL OF THE INSTITUTION OF STRUCTURAL ENGINEERS

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All Editorial communications should be addressed to The Editor, The Structural Engineer, 11, Upper Belgrave Street, S.W.1. All MSS. should be typewritten and algebraic signs and formulae be indicated distinctly.

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The President, 1960-61

The Institution welcomes as its President for the Session 1960-61, Lt.-Colonel G. W. Kirkland, M.B.E.(MIL.), M.I.STRUCT.E., M.I.C.E., who takes office on the 6th October.

Lt.-Colonel G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E.

George William Kirkland was born at Richmond, Surrey, in 1906. He was educated at Northallerton Grammar School in Yorkshire and later at Liverpool Collegiate School, and Blackburn Grammar School, Lancashire and began his professional career by apprenticeship to Dorman, Long & Company Ltd., at Redcar. This was followed by experience in mechanical, hydraulic, civil and structural work with various commercial and professional firms, also in tin and gold mining as Assistant to F. W. Payne & Son, Consulting Engineers. In 1934, he joined the late R. Travers Morgan as Assistant and from 1937 until he was commissioned in the Royal Engineers he was Chief Assistant to Mr. L. Scott White, O.B.E. It is interesting to recall that both Mr. Travers Morgan and Mr. Scott White have also rendered distinguished services to the Institution. Mr. Travers Morgan was a Member of Council for many years until his death in 1940 and Mr. Scott White is a Past President.

Colonel Kirkland served in the second World War in the British Expeditionary Force, the Middle East Force and in Paiforce as Engineer Staff Officer (Works). He was responsible for the design and administration of 5,000 miles 'Aid to Russia' road construction, including permanent bridging and other works and was awarded the M.B.E.(Military Division).

On his release from the Forces in 1945, he rejoined Mr. Scott White in the firm of R. Travers Morgan and Partners and was taken into partnership. In the course of his practice he has specialized in structural steelwork, reinforced and prestressed concrete construction, multi-storey buildings, roads, bridges and airfields and he is specially interested in foundation problems of all kinds.

He was elected an Associate-Member of the Institution of Structural Engineers in 1939 and a full Member in 1949. Two years later he was elected to the Council and has held office as Honorary Curator, Honorary Editor and, since 1956, as Vice-President. He has taken an active interest in every aspect of the Institution's work, having served on the standing Committees dealing with Finance, Membership, Bye-Laws, Practice and Special Awards and also on the *ad hoc* Committee on Structural Safety and the Aluminium Development Association Research Scholarship Committee. He has represented the Institution on the Ministry of Education Joint Committee on Higher National Diplomas and Certificates in Building, the Parliamentary and Scientific Committee and the South-East London Technical College.

Colonel Kirkland has contributed several papers to the Institution, the most recent being one on the subject of tall buildings which he read earlier this year. In 1959 he gave a paper on 'Urban Motorways' and in 1951 he presented a paper in collaboration with Mr. A. Goldstein on 'The Design and Construction of a Large Span Prestressed Concrete Shell Roof.' For this paper, the authors were awarded the Institution Bronze Medal and the London Prize for the Session 1950-51. In collaboration with Mr. L. E. Kent, Colonel Kirkland also contributed a paper on the 'Construction of Steel Framed Buildings' to the Jubilee issue of 'The Structural Engineer.'

A man of many interests, he is a Past-President of the Prestressed Concrete Development Group, a Member of Council of the Institution of Royal Engineers, a Fellow of the Royal Society of Arts, a Joint Honorary Secretary of the Parliamentary and Scientific Committee, a Member of Council of the Institute of Arbitrators and a Member of the Institution of Highway Engineers. He holds the rank of Honorary Lieutenant-Colonel (late R.E.). Colonel Kirkland presented the Cantor Lectures to the Royal Society of Arts in 1958 on Prestressed Concrete and lectured on this subject also in Edinburgh, Glasgow and Belfast. He was an Assessor for the Elevated Motorway Competition promoted by the Prestressed Concrete Development Group and headed the United Kingdom Delegation to the World Conference on Prestressed Concrete in San Francisco in 1957 and the International Conference on Prestressed Concrete in Berlin in 1958. He is the

Continued on page 328

Structural Considerations of a New Animal Feed Mill in Belfast, Northern Ireland *

by Edward V. Finn, M.I.Struct.E., A.M.I.C.E.
and
Kenneth F. Shadbolt, A.M.I.Struct.E.

Synopsis

A practical paper describing the new animal feed mill which has just commenced operation in Belfast, Northern Ireland.

It describes the selective use of structural steel, reinforced concrete and prestressed concrete for the various ancillary buildings incorporated in the project and the reasons for their use.

It claims to show how Engineers can produce 'purpose made' buildings with clean and pleasing lines, by the careful and economic use of conventional building materials, without embellishment of any kind, save that which can be obtained by the judicious use of formwork.

Introduction

In August 1957, four British manufacturers of animal feeding stuffs—The British Oil & Cake Mills Ltd., Crosfields & Calthrop Ltd., Lever's Cattle Foods Ltd., and R. Silcock & Sons Ltd.—announced their intention to erect a factory to manufacture jointly their products in Northern Ireland. A new company, Associated Feed Manufacturers Limited, was formed to operate the mill on behalf of the four companies, who will, however, retain their respective identities in developing the trade in their products, which will be manufactured to their own individual formulae and special process requirements. A former flax spinning mill was acquired for this purpose in York Road, Belfast, formerly the property of the York Street Flax Spinning Company.

The client appointed Messrs. Frederick S. Snow & Partners, Consulting Engineers, to co-ordinate and design all the building works except the silo building which was designed and constructed by Messrs. Christiani & Nielsen Ltd. as a separate contract. Messrs. Munce & Kennedy of Belfast were appointed by the Consultants as Architects for both the office block and personnel and labour office so that the buildings on the York Road frontage would be similar in character. Messrs. J. D. Wilson & Partners of Bristol produced all mechanical engineering drawings to designs prepared by the technical staff of the Associated Feed Manufacturers Limited.

Because of the nature of the material produced, special consideration had to be given to the details of the buildings to ensure clean construction. Every precaution has been taken by those responsible for the mechanical installation to minimise dust and independent dry and wet aspiration systems have been installed throughout the plant with this object. Nevertheless, the very nature of the process gives rise to a certain amount of dust and the claddings were designed with this in mind. The scheme chosen for the cladding of the industrial buildings incorporated asbestos cement sheeting, lined where necessary, patent glazing and engineering bricks for the lower 6 ft. 6 in. in locations liable to damage. Concrete buildings were left unclad. Internally, these buildings have a high standard of finish and tiling was extensively used in all production areas.

Design work commenced in January 1958, and limited production commenced in May 1960.

Description of Works (Fig. 1)

The new project comprises :—

(a) Roads, hardstandings and civil support work.
(b) Intake warehouse.
(c) Silo building.
(d) Production building.
(e) Finished products warehouse.
(f) Boilerhouse and chimney.
(g) Edible oils building.
(h) Engineering shop and stores.
(i) Office block.
(j) Personnel and welfare block.
(k) Sub-station, fuel farm and weighbridge.

Roads, Hardstandings and Civil Support Work

Extensive existing foundations were present, together with existing drainage and culvert systems.

The soil near the surface was inferior fill material and sleech which was laid on sandstone and the depth from ground level to the top of the sandstone varied from approximately 5 ft. to 50 ft., the smaller depth being in the area of the office and the greater depth being in the south-east corner near the boilerhouse. Trial bores and holes were taken in an early stage of the development, and due to the varying depth of the sandstone, and the presence of existing bases, the type of foundations used varied throughout the site. The varying nature of the sub-base and the quantity of hardcore arising from the demolition work, dictated the use of flexible construction for the majority of the roads and hardstandings with the exception of those portions where it was known that vehicles would stand for long periods. This method of construction will allow the pavement to settle slightly without damage and in the unlikely event of large depressions forming in the future, the surface can easily be made good. The construction chosen consisted of 3 in. two coat asphalt with ¾ in. chippings rolled into the surface ; 6 in. dry lean concrete, laid on a blinding of 2 in. quarry waste, which, in turn, was laid on a minimum depth of 12 in. consolidated hardcore. The reinforced concrete hardstandings consist of 10 in. Quality

* *Paper to be read before the Institution of Structural Engineers* 11, *Upper Belgrave Street, London, S.W.*1, *on Thursday, 13 October*, 1960, at 6 p.m.

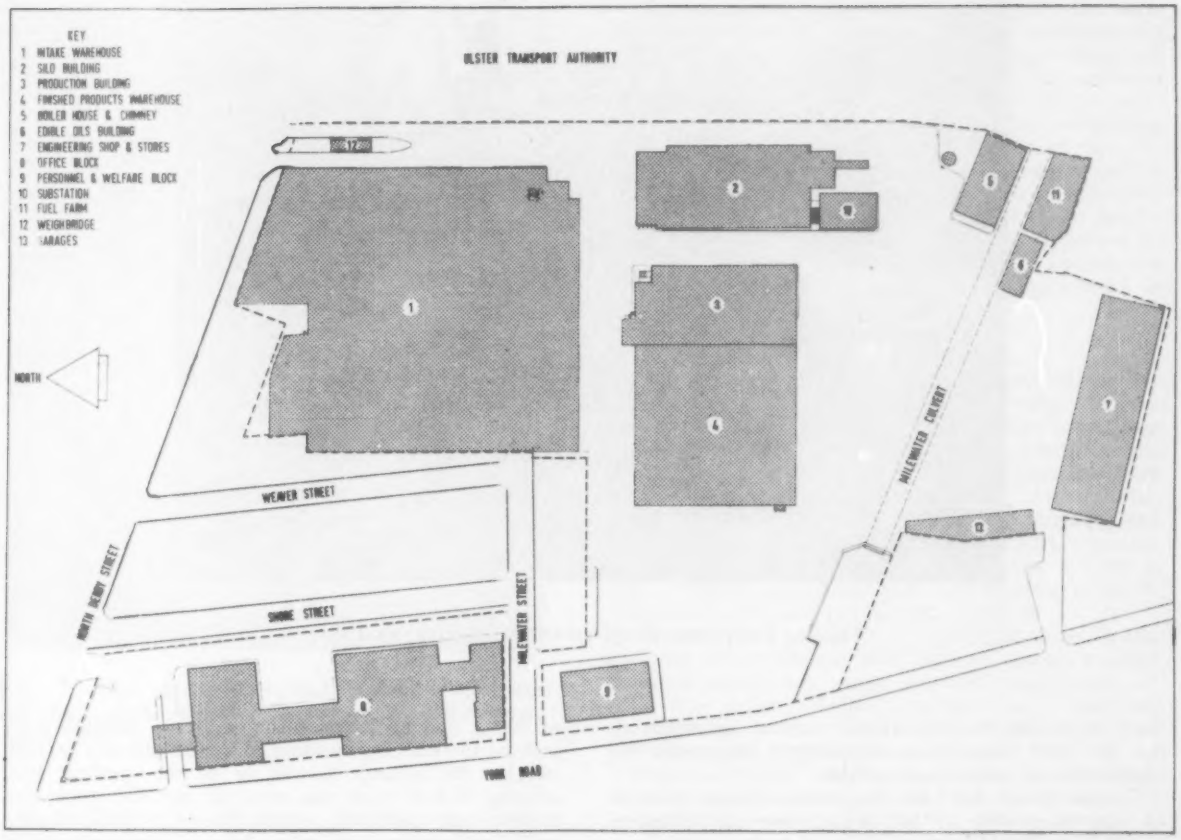

Fig. 1—Site layout

'C' concrete (3,000 lb./sq. in. at 28 days) with 10·10 lb./sq. yd. mesh in the top. The bay widths vary between 10 ft. and 15 ft. depending on the location. Dowelled joint construction was incorporated using 1 in. diameter mild steel bars at 12 in. centres in the expansion joints, and $\frac{3}{4}$ in. diameter mild steel bars at 12 in. centres in the contraction and longitudinal joints. The 10 in. pavement was laid on 0·0025 in. polythene sheeting on 1 in. sand blinding on consolidated hardcore.

Surface water drainage in the area is an important feature of design as the Milewater River runs through the south-east portion of the site. This has been culverted for 330 ft. using two 6 ft. diameter pipes. These are completely encased with concrete and inspection manholes provided. A short reinforced concrete section was used between the new pipe section and the railway bridge. An expansion joint was formed at this junction to allow for differential movement between the bridge and the new work. (Fig. 2).

A grille was constructed upstream in the open section to prevent branches and other large objects from blocking the culvert. Surface water from the adjacent area is drained into the new culvert above flood level. At one stage, the whole of the open portion of the Milewater was to be culverted, but it was decided to leave the remainder of the work until later. It was necessary to raise the finished ground level of the site to prevent flooding in the future, and a wall was constructed around the open portion. Careful consideration was given to the levels on the whole of the site and as part of the re-grading work Milewater Street was raised in one section. The re-grading level had to take into account future development in the area. The northern portion of the site is drained into the main drain in the north-east corner. Because of the depth of this main drain, back drop manholes were used. Soil drainage discharges into the main drain in the north-east corner of the site or into York Road where convenient.

A large underground services duct links the fuel farm and the production building, with a connection to the boilerhouse. This duct enables the various services, including molasses and steam, to be linked to the production building in a neat manner, thus avoiding the many overhead services normally associated with this type of project. A number of designs were considered, but in this location it was found economical to construct the walls in brickwork on a reinforced concrete slab and to use precast concrete roof slabs supported on removable R.S.J's. These were arranged so that if necessary they could be removed to gain access to the entire length of the culvert, although normal maintenance work or cleaning could be carried out within the access provided through manholes. It was decided not to tank the duct, but to drain it and falls were provided. Drained service

Fig. 2.—Early stage of culvert construction—July 1958

ducts were also provided to the various buildings on the site and these were constructed of brickwork roofed with precast concrete slabs.

The insulation used for the steam services was of an external quality so that the presence of dampness in the ducts would be of no consequence.

Intake Warehouse

An existing single storey steel framed warehouse, clear eaves height 12 ft. was modified to become the intake warehouse wherein are stored the raw materials to be processed. The modifications included reducing the width to the east by shortening the existing trusses and re-erecting on the new stanchion line, and extending the length to the south, the new brickwork being constructed to match the existing brickwork. This extension has a clear eaves height of 20 ft., and an investigation into the possibility of jacking the existing building to this height was carried out but the scheme proved to be too costly. The existing R.P.M. roof was repaired and the extension roofed with asbestos cement sheets. Other work included a supervisor's office, bag re-claiming section, toilets and locker room for female staff employed in this department. Part of the new floor was surfaced utilizing 2 in. paving slabs recovered from the existing floor. A reinforced concrete slab was constructed for the remainder. (Fig. 3). Tanked ducts were incorporated in the floor to house the drag conveyors feeding the elevator tower, in the south-east corner of the building. This tower is steel framed, clad in asbestos, and houses elevators raising material to a height of approximately 35 ft. to the conveyor gantry feeding the silo building. (Fig. 4).

Fig. 3.—Interior of existing portion of intake warehouse—January, 1960.

Fig. 4.—East wall of intake warehouse showing elevator tower—May 1960.

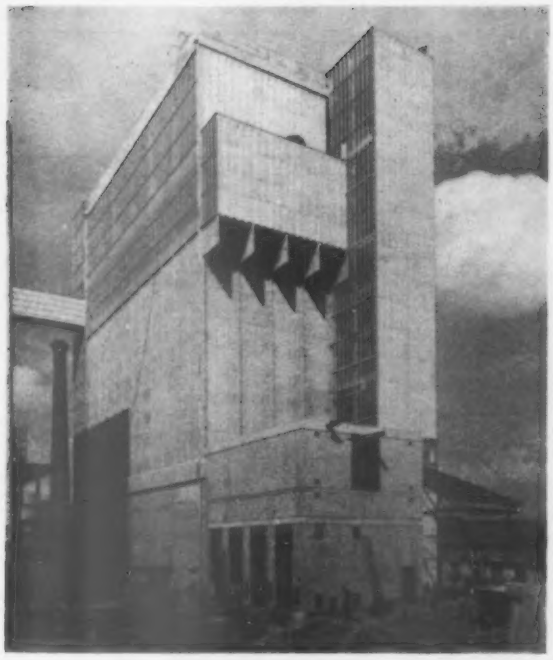

Fig. 5.—Silo building viewed from south west showing south elevator tower and sub-station—April 1960.

A steel-framed weighbridge office has been located in the centre of the road from the main access to the site off North Derby Street, with provision for two weighbridges, although only one has been installed. A Lamson tube installation links this building with the main despatch office situated in the finished products warehouse.

Silo Building

This building contains the heart of the process providing both conventional storage of unground raw materials in bulk and also pre-grinding equipment and automatic batch and continuous blending plants. Raw materials are conveyed by mechanical means into this building either in bulk form as received or reclaimed from bag storage in the intake warehouse. It is situated by the eastern boundary, adjacent to the railway and is 111 ft. long, 55 ft. wide and 125 ft. high. It has a capacity of approximately 5,000 tons with 100 separate bins. The two sizes of bins used are approximately 9 ft. × 9 ft. and 9 ft. × 4 ft. 6 in. Steel framed tower buildings are provided at the north and south ends. (Fig. 5).

These buildings are founded on Vibro piles, some of which had to be re-driven when tests proved that the safe working load of 60 tons had not been achieved. It is interesting to note that the first piles of a group were the ones that failed. The pile caps, pits, and other work within the ground and the silo itself were designed and constructed by Messrs. Christiani & Nielsen Ltd. The silo is a reinforced concrete structure incorporating precast post-tensioned panels forming the bin walls. These walls are 6 in. thick, and up to 20 ft. × 6 ft. 3 in. in size. They are stressed, using the Gifford-Udall system, with 0·276 in. wires.

The entablature floor, which supports the bin walls and hopper bottoms, is carried on reinforced concrete columns. Above the bins is a conveyor platform and this section is clad with patent glazing and asbestos cement sheeting to match the remainder of the work.

The steel towers, designed by the Consulting Engineers, provide access at both ends of the silo and house the main belt and bucket elevators used in the material conveying systems. A lift is incorporated in the north tower. A point of interest is the weight of the electric cables that have to be supported in this type of project where a high degree of automation is used. The weight of such cable groups can be in the order of 2 cwts. a foot run.

Production Building

Raw materials in their correct proportions by weight for each product batch are conveyed from the blending plant in the silo building to the production building and enter at high level to feed a row of mixers. After a controlled mixing period the other auxiliary processes cooking, extrusion, screening, cooling, etc., are carried out before re-elevation for weighing and bagging. The various processes mentioned are located in the vertical plane of the building and gravity is used for inter-stage connection. The building is 120 ft. long, 48 ft. wide and 113 ft. high. (Fig. 6).

The main structure is a semi-rigid steel frame designed in accordance with B.S.449, taking account of wind forces and using moment connections at main floor levels. A steel structure was chosen due to the large amount of machinery installed and to provide utmost flexibility for the future. (Fig. 7).

Universal rolled steel sections were incorporated, to the Authors' knowledge for the first time in Northern Ireland, and were found to be extremely useful, particularly for the columns. Teething troubles, however, were experienced, namely that the thicker members gave rise to splicing problems as allowance had to be made for the variation in the shape of the sections and the twist of some of the members. (Fig. 8). Also the steelwork contractor had to take great care in drilling site reamed holes for turned and fitted bolts, and in welding to thick flanges of universal sections and plated girders. This latter problem was largely overcome by the use of the automatic submerged arc welding process.

In order that future alterations of plant can be made, only the main members of the frame were cased for fire protection. After considering various forms of casing it was decided, in this instance, that concrete casings were preferable in order to minimize the risk of vermin or other pests lodging in concealed voids.

Apart from the ground floor, there are only three reinforced concrete floors, access to other parts of the building being provided by local platforms or galleries of chequerplate running longitudinally down the building. Quality B (3,750 lb/sq. in. at 28 days) concrete and high tensile cold worked deformed bars were mainly used.

The foundations consist of normal reinforced concrete bases with reinforced concrete columns to ground floor level. Recesses were formed in the reinforced concrete floor above the columns to conceal the steel stanchion bases.

As the sandstone was rising in this area and due to the presence of existing foundations, it was found that this type of construction was more economic than piles. Common foundations were used for the line of columns and stanchions between the finished products warehouse and the production building.

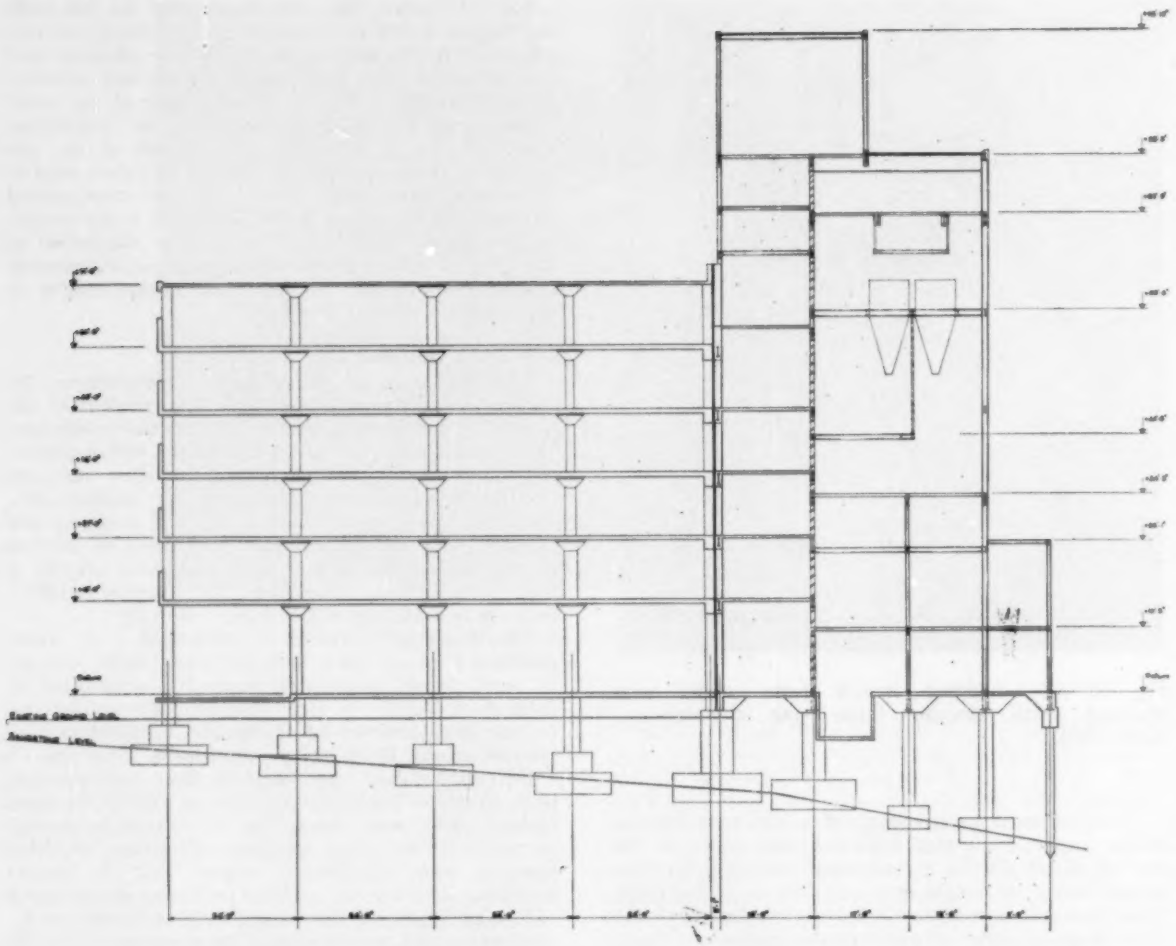

Fig. 6.—Section through production building and finished products warehouse.

The external cladding consists either of asbestos cement sheeting or double patent glazing. (Figs. 9 & 10). Normally, double glazing consists of wire cast glass internally and $\frac{3}{16}$ in. rolled glass externally except to production and access areas where Georgian plate glass is used in lieu of the cast glass. In these locations a curb and handrail is provided. The asbestos cement sheeting is supported on light framework which was flame cleaned, primed and treated with a non-oxidising paint, a material found to be ideally suitable where protection is required to steelwork which is not accessible. This steelwork also supports the sealed inner lining which consists of flat asbestos panels in areas which are not tiled. The internal asbestos is painted and in conjunction with the tiles at floor level, a very neat and clean internal wall finish has been achieved. (Fig. 11). Considerable care has been given to the internal finishes of the building, and the main working floors are paved with dense vitrified tiles, which are capable of being cleaned should oil and grease be deposited during maintenance operations. (Fig. 12). When considering the remainder of the internal finishes and details, it was decided to protect all access doors in vulnerable positions with steel angles or channels. Some of these doors were, of necessity, fire or blast doors of suitable heavy construction. These were located in walls of either brick or reinforced concrete which were plastered and painted or tiled.

The roof is constructed of asbestos reinforced decking and is weatherproofed with $\frac{3}{4}$ in. two coat asphalt. The pent house on the roof is clad to match the remainder of the building. A travelling cradle rail is provided at roof level above all areas of glazing.

Dust explosions are a potential hazard in the manufacture of animal feeding stuffs, as indeed they are in all processes which involve grinding and screening of certain materials. Our clients have attempted to meet all the recommendations of the Factory Inspectorate in this matter and all parts of the process potentially hazardous have been confined in certain areas which are separated from the rest of the buildings and plant with blast-proof walls. Generally the basic recommendations of one square foot of "weak-link" wall for every 20 cu.ft. of volume has been observed in all nominated blast areas. These "weak link"

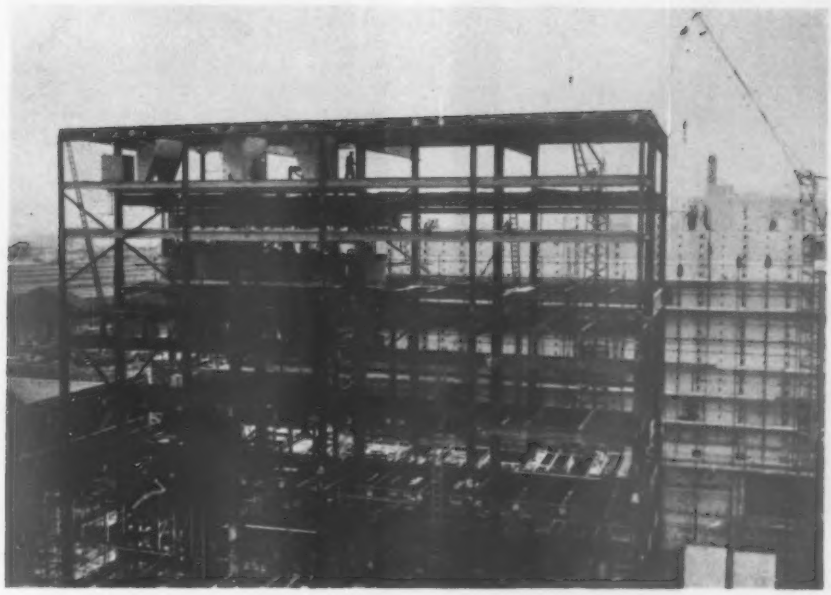

Fig. 7.—Production building steel frame—August 1959

Fig. 8.—Typical joint connection on production building—September 1959.

Fig. 9.—East wall of production building showing gantry joining silo building—January 1960.

Fig. 10.—South wall of production building showing adjoining finished products warehouse—July 1960.

walls are external walls and are constructed with light-weight materials which offer little resistance to explosion blast.

Very large volumes of air are used in the cooling of products and also in the wet and dry aspiration systems which run throughout this building. All air used in this manner is exhausted to a common horizontal and vertical exhaust duct. This duct was constructed of galvanised sheet metal which was lined with 1 in. thick cork insulation protected with $\frac{1}{8}$ in. limpet board. This insulation is necessary to minimise condensation.

Finished Products Warehouse

The finished products warehouse is a six storey building 71 ft. in height overall and approximately 120 ft. square on plan, adjacent to the production building. Its function is to receive and store bagged feed from the production building and provide quick and flexible loading off points for lorries. (Fig. 13).

The main line of bagging machines is situated at fifth floor level and filled bags are distributed vertically through the building using a system of part spiral chutes and steel band conveyors on each of the intermediate floors. Bags are taken off at their required floor from the steel band conveyors, loaded on to half-ton stillages, and removed by stillage truck to the storage bay on the floor. On the ground floor a system of one-ton pallets stacked four high is used for specially selected products.

The building was divided into five bays in each direction with columns at 24 ft. centres both ways. The ground floor is completely open apart from the staircase and despatch office, allowing lorries to park, two per bay, directly under the first floor in the perimeter bays. (Fig. 14). The spiral chutes are generally located in the first interior bays. The design superimposed load for each floor was 3 cwt. per sq. ft.

The superstructure was designed in accordance with CP.114, using flat slab construction and trapezoidal column heads, without drops. The upper floors are 10 in. thick reinforced concrete slabs with $1\frac{1}{2}$ in. non-monolithic granolithic generally and steel faced flags in the trucking runs. The roof is 9 in. thick reinforced concrete slab with lightweight screed laid to falls. It is weathered with $\frac{3}{4}$ in. asphalt and the screed has been vented. The columns which vary from 27 in. square to 18 in. square in three lifts, have column guards to a height of 4 ft.

Difficulty was experienced in detailing the floor slabs since the size of the chute holes i.e. 5 ft. 6 in. diameter, was the maximum permitted by the code of practice and the client required the holes to be located as far into the column strip intersections as possible. Due to the ensuing congestion of reinforcement the contractor had to exercise great care when placing the concrete, but produced a satisfactory job. The holes were trimmed with $\frac{1}{4}$ in. mild steel plate lugged back into the concrete, the lugs, in turn, being welded to 1 in. diameter ring bars. Apart from transmitting local load to the concrete this arrangement will assist in re-distribution of local concentrations of stress. (Fig. 15).

The ground floor was designed as suspended since the site was backfilled to some considerable depth in this area. Due to variation in external levels, it was possible to incorporate a loading bay in the interior of the building on the north side. Beam and slab construction was adopted for this floor.

The columns are founded on sandstone at depths up to 18 ft. below ground floor level and the foundations consist of small reinforced concrete pads on mass concrete blocks. The permissible bearing pressure on this sandstone is 5 tons/sq. ft.

Fig. 11.—Internal view of production building nearing final stage of plant erection.

Fig. 12.—Internal view of completed silo building showing typical interior of all operating floor finishes.

Due to the stacking height of 6 ft. the perimeter wall consists of an upstand beam to this height with continuous bands of glazing between the top of the beam and slab soffit. The glazing has opening lights to permit through-draught, and is controlled from two points on each floor. From the photographs it can be seen that this gives an elevation that is simple yet pleasing. (Figs. 10 & 13). The finish to the exterior face of concrete was untreated sawn formwork with emphasis on the joint between the boards. (Fig. 16).

To cater for possible extensions, starter bars for future continuation of the slabs were bent up into the upstand beam on the south and west walls.

One staircase has been provided, constructed in reinforced concrete with insitu stringers with polythene joints and pre-cast treads and risers. This method of construction was adopted to allow for possible re-positioning of the staircase when extension takes place.

Alternative escape is provided by two staircases in the adjacent production building.

The following mixes of concrete were used :—

(1) Columns—Quality A, 4,500 lb/sq. in. at 28 days.

(2) Slabs and beams—Quality B, 3,750 lb/sq. in. at 28 days.

(3) Foundations—Quality C, 3,000 lb/sq. in. at 28 days.

Deformed cold worked high tensile steel (30,000 lb/sq. in.) and mild steel (20,000 lb/sq. in.) were used where most advantageous.

A 1½ in. expansion joint was provided between this warehouse and the production building and the floor joint was protected by steel angles with a sliding plate bridging the gap.

Boilerhouse and Chimney

To provide steam for processing purposes, space heating and domestic hot water for canteen, laboratory, toilets, etc., a boilerhouse has been erected in the south-east corner of the site. This building contains two automatic oil-fired package boilers each capable of delivering steam at an initial pressure of 150 lb. per sq. in. The boiler flues are connected to a horizontal flue which connects to the concrete chimney. To

Fig. 13.—Finished products warehouse viewed from north west showing relation to other buildings on site—July 1960.

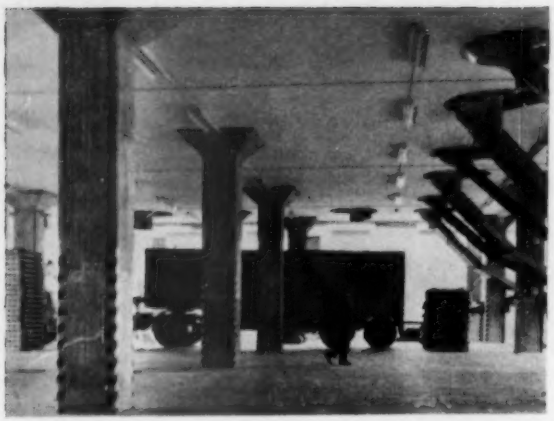

Fig. 14.—Ground floor of finished warehouse—July 1960.

ensure that the fumes will be well above the silo and processing buildings, the height of the chimney is 150 ft. (Fig. 17).

The boilerhouse is founded on Prestcore piles, and the depth to the sandstone was in the order of 45 ft. This is a single storey building, and incorporated in the ground floor slab are many ducts and plinths. The compressor house on the east also forms part of this building. A blowdown pit was constructed as part of the floor, but immediately outside the building, and this drains into the Milewater Culvert previously described. The blowdown pit was vented to a point above roof level. The building itself is a steel framed building clad in asbestos and patent glazing to match the production building. The same kind of engineering bricks were used to a height of 6 ft. 6 in. above ground level to match the remainder of the engineering structures on the site. Steel angle lattice girders, carried on stanchions at 10 ft. 6 in. centres were used to support the asbestos reinforced decking which was weatherproofed with $\frac{3}{4}$ in. asphalt. Quality C concrete was used for the foundations and plinths with mild steel reinforcement.

The reinforced concrete chimney is adjacent to the boilerhouse and is lined throughout with acid resisting bricks. The internal diameter at the base of the concrete shell is 8 ft. 7 in. and at the top 6 ft. 7 in. the exterior being formed of 16 equal facets. Quality B concrete was used with mild steel reinforcement, and the thickness of the concrete at the base is 10 in. and at the top 5 in. Two lightning conductors are provided and phosphor bronze dowels are built into the outer face for the erection of ladders for inspection and maintenance purposes. A sloping ash chute is built in the bottom of the chimney at ground level, the height of the flue entry being 15 ft. Access is gained to this chute by a steel door and provision has been made for the acid laden water formed in the chimney to drip in front of the chimney itself and be drained away from the face of the chimney to prevent staining. Acid resisting paving bricks are used on the sloping chute.

A feature of the chimney construction is that the lining bricks are supported every 25 ft. on corbels. A 2 in. cavity is provided and air holes are located at the top and bottom of each 25 ft. lift to ensure free circulation of air. The details have been arranged so that the acid laden moisture does not drip down the cavity between the chimney shaft and linings. This

Fig. 15.—Completed intermediate floor of finished products warehouse.

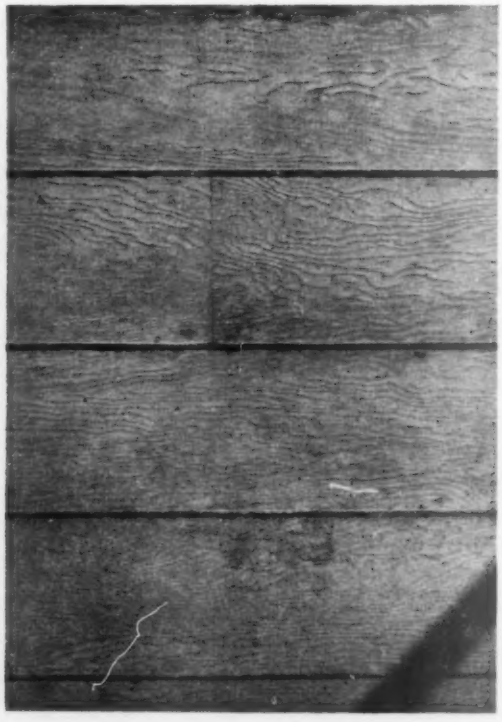

Fig. 16.—Close-up view of concrete finish of waterhouse.

Fig. 17.—View of south east corner of site showing boilerhouse, chimney, fuel farm and edible oils building—July 1960.

detail was used on a similar chimney constructed four years ago, which following a detailed examination was found to be satisfactory. The horizontal flue is an independent shaft which is supported on a steel trestle, an expansion joint being provided so that it imposes no vertical or horizontal load on the chimney itself. This shaft is constructed in steel and forms part of the mechanical plant contract. (Fig. 18).

Edible Oils Building

As an additional facility, an edible oils building has been provided which is a steel framed two storey building 44 ft. × 17 ft. The drums of solid vegetable or animal fat are hoisted externally to the first floor level and positioned on steam heated grills for pre-heating. For final melting electro thermal mufflers are used, the melted fat running into storage vats on the ground floor and thence pumped to the production building for process purposes. This building is founded on Prestcore piles and has a reinforced concrete floor. The steel frame is clad externally to match the boiler-house except that in this case the entire north elevation is glazed above the 6 ft. 6 in. engineering brick wall. The roof consists of a reinforced concrete slab with $\frac{3}{4}$ in. asphalt finish. Considerable natural ventilation is provided at first floor level and roof ventilators are incorporated. Dense vitrified floor tiles are used, as in the production building. Internally where there is no glazing, the building has a brick lining which is rendered and painted. To facilitate the handling of the drums, a lifting beam runs the full length of the building and extends at one end through double doors. Access is gained to first floor by an external steel staircase on the south side.

Engineering Shop and Stores

The company will operate its own engineering department for the repair and maintenance of its highly mechanised and automated plant and equipment. A fully equipped engineering shop and stores has been constructed at the southern end of the site. The building is 168 ft. long, 48 ft. wide and 28 ft. high to top of parapet level and a 2-ton electrically operated overhead crane is installed capable of traversing the whole length of this building thereby serving all departments. (Fig. 19). Welded rolled steel joist portal frames are used at 15 ft. centres and these are designed as fixed at the bottom. Bracing is incorporated to take account of the horizontal surge of the crane; this and other secondary steelwork is bolted and not welded in position. Steel portals are founded on individual reinforced concrete foundations using a permissible bearing pressure of only $\frac{1}{2}$ ton/sq. ft. This was found reasonable as the column loadings are relatively light. The ground floor rests directly on the sub base, the sub base being brought to the correct level by hardcore which is blinded with sand and covered with polythene.

The external walls consist of engineering bricks to 6 ft. 6 in. level with industrial windows and asbestos cement sheeting above. The roof to this building is asbestos cement sheet sandwich construction with fibre glass infill. The walls of the building are lined internally with flat asbestos sheet panels fixed to lightweight galvanised metal tees secured to the steelwork. Security steel mesh encloses the store areas.

High level power driven ventilators are provided and it is possible to use these separately, thus giving controlled ventilation during summer months. The building is space heated throughout the store and working areas, and heated with radiators in the offices. Toilet facilities are provided.

Fig. 18.—View of lower portion of chimney showing flue entry—October 1959.

Fig. 19.—View of engineering shop and store in background showing early stage of construction of edible oils building—May 1960.

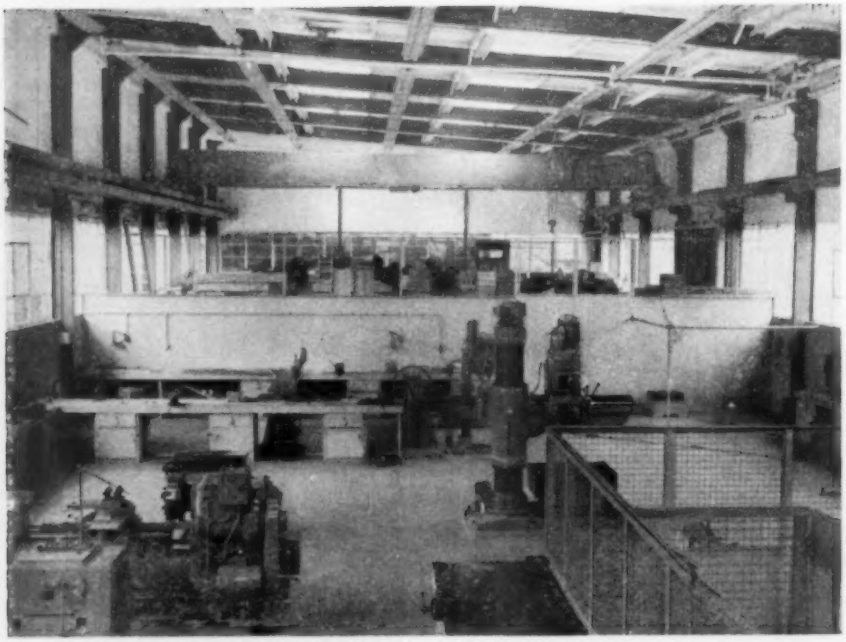

Fig. 20.—Interior of engineering shop and store—July 1960.

The offices and stores are formed with patent partitions to suit. The ground floor is surfaced with metallic aggregate mono-granolithic screed and the internal brick wall, constructed to cill height, is painted. (Fig. 20).

Office Building

The main administration offices are situated on the York Road frontage. This building is divided by expansion joints into three parts ; the administration block on the north end ; the laboratory block and catering facilities on the south end ; and the link block between. (See Fig. 13).

This building is founded on normal reinforced concrete foundations approximately 6 ft. deep with ground beams to carry all external brickwork. The depth to the sandstone on the site is the minimum in this area, and all excavations uncovering the sandstone were immediately blinded with 3 in. concrete as it was found that the sandstone deteriorated on exposure and became excessively affected by water.

Quality B concrete was used for the frame and hollow pot floors. The spans vary betweeen 13 ft. 4 in. and 23 ft. although the main grid is 16 ft. 8 in. by 20 ft. The ground floor is a 6 in. thick Quality C concrete non-suspended floor reinforced with fabric reinforcement.

External cladding of this building was designed by the Architect and consists mainly of 2 in. bricks with treated aluminium windows. The roof is generally two coat $\frac{3}{4}$ in. asphalt laid on screed but three layers built-up bituminous felting is used for the administration block as this has been designed to accommodate another floor in the future.

The administration block is 76 ft. by 52 ft. and accommodates managerial and general offices. The link lock is 62 ft. by 28 ft. wide accommodating further offices and toilets.

The adjacent laboratory block 83 ft. by 52 ft. on plan has on its ground floor a well equipped kitchen and canteen. Above this on the first floor is the laboratory complete with all associated facilities. 40 ft. clear span monitors have been provided above the main laboratory to give a high standard of natural light.

A single storey annexe to the laboratory block houses the welfare and health facilities and incorporates consulting, treatment and rest rooms together with other necessary accommodation. A sub-station for the Belfast Electricity Department has been incorporated within this section.

Sub Station

To house the main transformer, and medium voltage and high voltage switchgear, a sub-station has been constructed immediately south of the main silo building. This is, in effect, a three storey building, 40 ft. wide, 20 ft. long and 28 ft. high supported on piles.

Instead of constructing the ground floor with many ducts and pits a tanked pit is formed over the area of the building and interior walls are built to suit. The ground floor is then supported by these walls and the transformers and H.T. switchgear are situated at this level. Above these there is a services duct floor 3 ft. in height to house the numerous cables and immediately above this is housed the client's M.V. switchgear. The one building houses both the client's and the Belfast Corporation Electricity Department's equipment. Above ground floor level, load bearing concrete block wall construction is used, together with both insitu and precast concrete units for the roof and cable duct floors. The roof is flat with lightweight screed topping laid to falls finished with $\frac{3}{4}$ in. thick asphalt. A high level lifting beam is situated beneath the soffit of the roof to provide easy handling facilities for the M.V. switchgear at this level. This extends beyond the doors at the south end of the building.

Fuel Farm

The fuel farm is also situated in the south east corner of the site over the former open portion of the Milewater. Within the confines of this structure are housed the molasses and oil tanks. Large quantities of fill material were used to bring this to the correct level. When the unsuitable sleech material was excavated, the timber piles, which were supporting an adjacent building, were covered with clay to maintain their moisture content.

A draining valve, which is normally kept closed, is provided in the reinforced concrete base slab, and the tanks enclosed on four sides by an 8 ft. high bund wall. A pit is also provided so that, should there be a leak, the oil or molasses can be pumped out into suitable vehicles and not drain into the surface water drainage system.

The tanks themselves rest directly on a 2 in. dense tar bed. This enables the tanks to settle on the foundations, the tar protecting the bottom of the tank. Access ladders are provided and the concrete structure is designed using Quality C concrete with mild steel reinforcement.

Personnel and Welfare Office

A two-storey building 67 ft. × 27 ft. has been constructed to accommodate the personnel department, time office and various employees' amenities. These include locker rooms, showers, toilets and mess-room for night workers etc. The building is of reinforced concrete construction and has been architecturally designed to match and blend with the main office block.

Acknowledgements

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A New Method for Predicting the Natural Frequency of Foundation-Soil Systems

by

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Synopsis

After giving a brief review of the existing methods of determining the natural frequency of foundation-soil systems, this paper presents a new method of predicting the natural frequency. Its validity is established by comparison with the results of Pauw, Converse and Eastwood. The new method suggested is easy to apply, and is simple in form.

1. Machine Foundations

The foundation carrying a machine and resting on the soil may be replaced for purposes of convenient analysis by a single equivalent mass supported by a perfectly elastic spring. For simplicity in calculation, the soil is replaced by a spring and this soil-spring analogy is well known in soil dynamics. Such a single-mass-spring system has six degrees of freedom of which three are rotational and the other three are translational. Consequently such a system has six natural frequencies.

The most significant point in the design of a machine foundation is the elimination of the possible occurrence of resonance caused by the coincidence of the operating frequency of the machine with the natural frequency of the foundation-soil system in any of the six degrees of freedom. Resonance must be avoided at all costs, as otherwise the amplitude of oscillation of the machine builds up and the violent oscillation of the machine impairs its efficiency and accuracy. As the operating frequency of the machine is fixed the only method open to the designer is to control the natural frequency of the foundation-soil system by suitable means in order to avoid resonance. Therefore, the initial problem that faces a designer is to predict the natural frequency of a foundation-soil system.

2. Natural Frequency of a Single Mass Spring System

The behaviour of a single mass resting on a weightless, linear spring and subjected to a single impulse may be described by the differential equation :

$m\ddot{x} + kx = O$,* where

m = mass

k = spring constant

x = displacement

$\ddot{x}$ = acceleration

The natural frequency of such a system is given by

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k}{m}} = \frac{1}{2\pi}\sqrt{\frac{kg}{w}} \quad . \quad . \quad . \quad . \quad (1)$$

where w is the weight supported on the spring which is assumed to be weightless.

* Any Book on Mechanical Vibrations for instance, by Den Hartog, McGraw Hill Book Co.

If the effect of damping is considered, the differential equation gets modified to :

$m\ddot{x} + c\dot{x} + kx = O$ where c = damping coefficient.

The corresponding expression for the natural frequency is

$$f_{nd} = \frac{1}{2\pi}\sqrt{\frac{k}{m} - \frac{c^2}{4m^2}} \quad . \quad . \quad . \quad . \quad (2)$$

In the soil-spring analogy, the soil is replaced by a spring which is by no means weightless. If the weight of the spring is also considered, and if the damping is neglected, the equation (1) may be written as

$$f_n = \frac{1}{2\pi}\sqrt{\frac{kg}{W_s + W_v}} = \frac{1}{2\pi}\sqrt{\frac{K^1 Ag}{W_s + W_v}} \quad . \quad (3)$$

where

W_v = weight of the foundation block and the machine,

W_s = weight of the spring i.e., the weight of the soil actively participating in the vibrations,

K^1 = modulus of dynamic sub-grade reaction given by $K^1 A = k$,

A = area of the base of the foundation.

Tschebotarioff[1] has pointed out that there is no clear cut line of demarcation to denote the part of soil which is actively participating in the vibrations of the foundation block. In the absence of more information, he gave W_s as the equivalent weight of the soil which is supposed to be participating in the vibrations.

3. Pauw's Analogy of the Foundation-Soil System[2]

Pauw's work in establishing the natural frequency is remarkable. In his work he makes the following assumptions :

i. that the Coulomb's law governing the shear strength of soils is valid

ii. that the modulus of elasticity of a cohesive soil is constant irrespective of depth

iii. that the modulus of elasticity of a non-cohesive soil increases with depth and is proportional to the effective depth (* See Appendix 1)

iv. that the Boussinesq equation is valid i.e., the soil is elastic, isotropic and homogeneous.

The dispersion of the surface load is further assumed to be uniform. Thus a uniformly distributed load at the surface of the soil is dispersed in the form of a truncated pyramid or cone according as the base is rectangular or circular at top. This assumption, although erroneous, is made for purposes of simplifying the evaluation of the effective value of the subgrade reaction and the error is not of much consequence.

The spring constant of the truncated pyramid or cone is calculated by first of all determining the surface deformation. It is given by an infinite integral

$$\delta z\,(o) = \frac{P}{\beta}\int_{o}^{\infty} \frac{dz}{(a + \alpha z)\,(b + \alpha z)\,(h + z)} \quad \cdot \quad \cdot \quad (4)$$

where a is the length of foundation block
b is the least width or d = diameter of circular base plate
z is the depth below the surface
p is the total load at the surface
$\frac{\alpha}{2}$ is the dispersion angle.

Utilising this, Puaw has arrived at the spring constants as

$$\left.\begin{aligned} K_z xy &= \frac{\pi}{4}\,\beta\, d^2\, r_z xy \text{—}(a) \\ K_z xy &= \beta\, b^2\, r_z xy \quad \text{—}(b) \end{aligned}\right\} (5)$$

(a) for circular base in non-cohesive soil
(b) for rectangular base in non-cohesive soil

in which β is a constant defined by the equation

$$E(z) = \beta\,(h + z) \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (6)$$

in which $E(z)$ is the elastic modulus at depth z, h is the equivalent height of surcharge. This constant β is found by dynamic tests.

In equation 5 :

$$\left.\begin{aligned} \frac{1}{r_z xy} &= \int_{o}^{\infty} \frac{dW}{(r+W)(1+W)(S+W)} \quad \text{for rectangular base} \\ &= \int_{o}^{\infty} \frac{dW}{(1+W)(1+W)(S+W)} \quad \text{for circular base} \end{aligned}\right\}$$

$$\cdot \quad \cdot \quad \cdot \quad \cdot \quad (7)$$

Where $W = \frac{\alpha z}{b}$ $S = \frac{\alpha h}{b}$ $r = {}^{a}/_{b}$ for rectangular base and $W = \frac{\alpha z}{b}$ $r = {}^{a}/_{b} = 1$ $S = \frac{\alpha h}{b}$ for circular base. This method of determining the spring constant is used by the authors in their new method as explained later.

4. Converse's Empirical Equation[3]

Converse has evolved an empirical equation to forecast the natural frequency of a Foundation-Soil system. His work at California Institute of Technology was mainly concerned with the investigation of the effect of running the vibrator at resonant frequency on the variations of density in the underlying sand bed. As a result of these investigations he arrived at an equation for the spring constant of the soil :

$$k = 44 \cdot 3W + 16000 - 27F \quad \cdot \quad \cdot \quad (8)$$

where W = Weight of vibrating body in lb.
F = Dynamic force in lb.

Therefore

$$f_n = \frac{\sqrt{g}}{2\pi}\sqrt{44 \cdot 3 + \frac{16000}{W} - \frac{27F}{W}} \quad \cdot \quad \cdot \quad (9)$$

Making the equation dimensionally consistent by a suitable combination of parameters, the final equation is given by

$$f_n = 3 \cdot 13\sqrt{840\,\frac{\rho}{G}\left(1 \cdot 64 - \frac{\sigma_F}{\sigma_w}\right) + 0 \cdot 18\,\frac{G}{\sigma_w \cdot R}} \quad (10)$$

where ρ = density of the soil in lb/cu. ft.
G = Shear modulus in lb/sq. in. as determined by the torsion cylinder technique
σ_F = Unit dynamic force in lb.
σ_w = Unit dead load in lb.
R = Radius of vibrator base plate in inches.

Converse has experimentally checked the validity of the method for various conditions and the present authors have applied their new method to the results published by Converse.

5. The New Method Based on the Bulb of Pressure Concept

The calculations of the natural frequency by Barkan[4] (at *VIOS*) took no account of the mass of soil vibrating. But the work done by Degebo[5] indicates that when a vibratory load acts on a soil, a certain mass of soil ranging from four to ten times the vibratory load participates in the vibration. Eastwood[6] has also pointed out that the natural frequency depends on the shape and dimensions of the loaded area, the load carried, nature of soil and the moisture content. Crockett[7] observes that the natural frequency is a function of the velocity of the Rayleigh wave in the soil. As the velocity of the Rayleigh wave depends on the density and the elastic modulus of the soil, Crockett's findings indirectly relate the natural frequency with the density and elastic property of the soil. Crockett and Hammond[8] have hinted in their publications that the soil inside a certain pressure bulb is probably participating in the vibrations and that the natural frequency is governed by the oscillational properties of this bulb. They had no experimental evidence to support this.

From the above it may be inferred that all the earlier investigators were inclined to think that a certain mass of soil was participating in the vibrations and was affecting the natural frequency ; but experimental evidence was not available to assess the mass of this soil.

This problem of assessing the mass of soil participating in the vibration is approached as follows :

The vibrating mass of soil is assumed to be contained by the boundary of a pressure bulb. For the purpose of simplicity in obtaining the pressure bulb, the area loading i.e., the load acting on any area is replaced by an equivalent concentrated load acting at the mass centre of the original area. If a concentrated load P is acting on the semi-infinite mass, then pressure bulbs of intensity 90 per cent of P, 80 per cent of P, 70 per cent of P etc., can be traced very easily. Even if an additional load of Q is added on to P, the same set of pressure bulbs indicate 90 per cent, 80 per cent, 70 per cent of $(P + Q)$ as indicated in Fig. 1. For

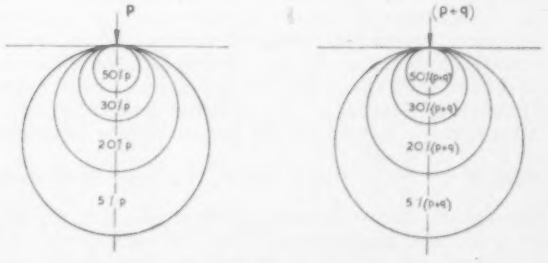

Fig. 1.—Pressure bulbs under point load p and (p+q) (Designated as percentage pressure bulbs). Note that the volume inside a particular percentage pressure bulb is constant.

Note :- p = static + maximum positive dynamic loads (assumed to be static).

convenience these pressure bulbs may be called " Percentage pressure bulbs." If it is assumed that a certain mass of soil contained within a certain percentage pressure bulb is oscillating under a known (static plus dynamic) load, the same percentage pressure bulb will not represent the soil oscillating if the static or dynamic or both are altered. While it is granted that a certain mass of soil is oscillating, the method of determining this mass on the basis of percentage pressure bulbs is not convenient. The alternative is to look for a pressure bulb of the same intensity and not for the same percentage as above.

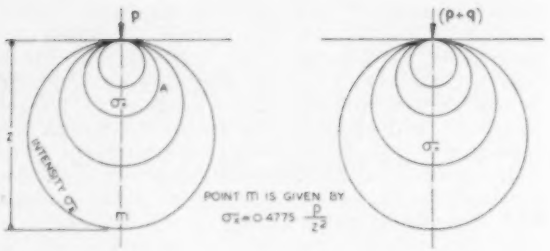

Fig. 2.—Intensity pressure bulbs. Note that the volume inside a particular σ_0 — bulb varies

For a certain surface load P concentrated at a point, let σ_0 be the intensity of pressure at any point A, (Fig. 2). A pressure bulb to represent an intensity of σ_0 can be easily traced. If the concentrated load were to change from P to $(P + Q)$, it is still possible to draw a pressure bulb of intensity σ_0. This pressure bulb will contain a larger mass of soil as is evident from Fig. 2. This is in keeping with the fact that the natural frequency is affected by the change in load (static or dynamic) and thus the further work is based on the concept of ' intensity pressure bulbs.' It should be borne in mind that these pressure bulbs are imaginary boundaries indicating the mass of soil participating in the vibrations and the boundary does not really exist within the soil mass. The designation ' Percentage ' or ' Intensity ' pressure bulb is also for clarity.

Crockett's work has indicated that the natural frequency is a function of the density of the soil. As the density of the soil at any site is a relatively reliable constant, the authors investigated on several ' intensity pressure bulbs ' which had some relation or other with the value of the density of the soil. If the initial density of the soil (before application of the vibrations) is S lb/cu. ft. the pressure bulbs corresponding to $\frac{S}{2}, \frac{S}{3}, \frac{S}{4},$ etc. lb. per sq. ft. were tried under the action of a concentrated load which is the sum of static and maximum positive dynamic load.

The masses of soil contained in these bulbs were calculated and investigations were carried out to see if any of the masses would give an answer to the mass of soil participating in the vibrations. The investigations reveal that the soil contained in the ' Intensity ' pressure bulb corresponding to the value of ρ lb per sq. ft. would give the answer as seen from the results tabulated.

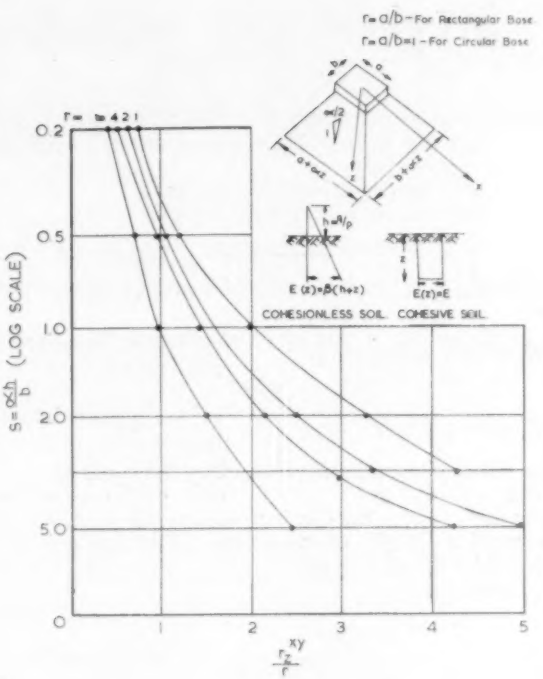

Fig. 3.—Equivalent soil spring constant for horizontal contact surface. (Extract from Pauw's Paper)

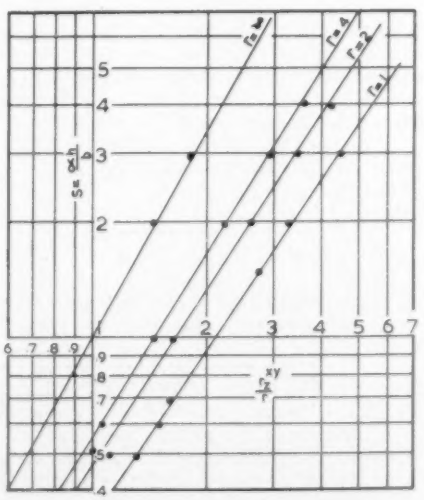

Fig. 4.—Fig. 3. Replotted on log-log scale

Table 1.—Application of the new method to the results published by Converse

Run	Diam. of Base plate in inches.	Dead load in lb.	Dynamic force in lb.	Resonant Frequency by Expt.	Calculated Frequency (By authors)	Remarks
No. 11—A	19·2	814	600	20·41	21·00	
No. 11—B	19·2	814	650	—	20·38	
No. 11—C	19·2	814	700	—	19·96	
No. 11—D	19·2	814	750	—	19·50	
No. 11—E	19·2	1070	800	19·17	19·50	
No. 12—A	19·2	610	300	22·91	24·60	
No. 12—B	19·2	865	300	—	23·80	
No. 12—C	19·2	865	450	—	22·30	
No. 12—D	19·2	1070	450	21·60	22·00	
No. 12—E	19·2	1070	600	20·40	20·80	
No. 12—F	19·2	1070	750	19·60	19·80	
No. 12—G	19·2	1070	800	—	19·50	
No. 13—A	19·2	1070	300	21·70	23·40	
No. 13—B	19·2	1070	450	20·40	22·00	
No. 13—C	19·2	1070	600	20·83	20·80	Used for finding β-value.
No. 13—D	19·2	1070	750	20·00	19·80	
No. 13—E	19·2	1070	800	19·16	19·50	
No. 14—A	19·2	548	300	23·30	24·30	
No. 14—B	19·2	548	450	22·08	22·20	
No. 14—C	19·2	781	450	—	22·20	
No. 14—D	19·2	781	600	20·41	20·70	
No. 14—E	19·2	1070	600	—	20·80	
No. 14—F	19·2	1070	750	20·40	19·80	
No. 14—G	19·2	1070	800	20·00	19·50	
No. 15	19·2	1070	750	—	19·80	
No. 16—A	19·2	1070	450	20·83	22·00	
No. 16—B	19·2	781	450	—	22·20	
No. 16—C	19·2	551	450	22·50	22·10	
No. 17	19·2	816	750	21·25	19·41	
No. 18—A	19·2	1070	300	22·16	23·40	
No. 18—B	19·2	1070	450	21·66	22·00	
No. 18—C	19·2	1070	600	21·25	20·80	
No. 18—D	19·2	1070	750	—	19·80	
No. 19—A	19·2	781	300	22·50	23·95	
No. 19—B	19·2	781	450	22·08	22·20	
No. 19—C	19·2	781	600	21·25	20·70	
No. 19—D	19·2	781	750	20·08	19·45	
No. 20—A	19·2	552	300	23·75	24·20	
No. 20—B	19·2	552	450	22·50	22·10	
No. 20—C	19·2	800	600	—	20·60	
No. 20—D	19·2	800	750	20·80	19·40	
No. 21	19·2	700	600	21·25	20·70	
No. 22—A	19·2	1070	300	22·50	23·40	
No. 22—B	19·2	1070	450	21·66	22·00	
No. 22—C	19·2	1070	600	20·83	20·80	
No. 22—D	19·2	1070	750	19·60	19·80	
No. 23—A	19·2	1080	300	23·33	23·20	
No. 23—B	19·2	1080	450	22·08	21·80	
No. 23—C	19·2	1080	600	21·25	20·56	
No. 24—A	15·7	1120	300	22·08	23·20	
No. 24—B	15·7	1120	450	21·25	21·25	
No. 24—C	15·7	1120	600	20·42	20·21	
No. 25—A	15·7	725	300	23·33	23·75	
No. 25—B	15·7	725	400	22·50	22·40	
No. 25—C	15·7	725	500	21·66	21·70	Used for finding β-value
No. 25—D	15·7	725	600	21·25	20·40	
No. 25—E	15·7	725	700	—	19·45	
No. 26—A	15·7	725	300	23·33	23·75	
No. 26—B	15·7	725	400	22·50	22·40	
No. 26—C	15·7	725	500	21·66	21·70	
No. 26—D	15·7	725	600	—	20·40	
No. 27	15·7	725	600	—	20·40	
No. 28—A	15·7	725	300	22·50	23·75	
No. 28—B	15·7	725	400	21·66	22·40	
No. 28—C	15·7	725	500	20·83	21·70	
No. 28—D	15·7	725	600	20·00	20·40	
No. 29	15·7	725	600	—	20·40	

In the equation $1, f_n = \frac{1}{2\pi}\sqrt{\frac{kg}{W_s + W_v}}$

where W_v (weight of the foundation plus maximum positive dynamic load) is known and W_s (weight of vibrating soil) is also arbitrarily found as the quantity of soil contained in the ρ lb per sq. ft pressure bulb. The only unknown quantity is the spring constant k. Pauw has worked out the spring constant of a non-cohesive soil at a particular site to be

$$\left.\begin{aligned} K_z xy &= \beta b^2 r_z xy \quad -(a) \\ K_z xy &= \frac{\pi}{4}\beta d^2 r_z xy \quad -(b) \end{aligned}\right\} (5)$$

(a) for a rectangular base; (b) for a circular base

where β is a constant given by $E(z) = \beta(h + z)$ (See Appendix 2)

Fig. 3 is a relationship given by Pauw from which it is possible to find $r_z xy$. This relationship is given as a semi-log relationship which for convenience is replotted to log-log scale as in Fig. 4. The advantage of this is that the graph in Fig. 4 covers higher values of S and $r_z xy$ the details of which are explained in the graph.

As the value of β is not given by Converse, it was worked out for each of the base plates of different diameters assuming the natural frequency given by Converse for a particular size of base plate to be correct. It was noticed in these calculations that the value of β varied with the diameter of the base plate and the exact relationship between β and the diameter of the base plate cannot be worked out with the available data. Having got the value of β for a base plate of a particular diameter, the value of the spring constant of the soil was determined.

Thus for other values of Table 1, all the quantities in equation (1) are known and the following is a typical calculation applying the new method to one of Converse's results :

Example : The base plate 12—F has been taken for illustration. The following is the information available:

a.	Diameter of base plate	19·2 ins.
b.	Dead load in pounds	1070
c.	Dynamic force in pounds	750
d.	Density of soil in lb/cu. ft.	88
e.	β Value assumed in lb/cu. ft.	150000

Step No. 1 :—

Total Load = 1070 + 750 = 1820 lb.

This load acts as a concentrated load on the surface Part.

Diameter of the bulb as found by the formula

$\sigma_z = \frac{Q}{z^2} \times I\sigma$ (Reference 9)

$88 = 1820/Z^2 \times 0{\cdot}4775$

$\therefore \quad Z^2 = (\text{diam.})^2 = 9{\cdot}86$

$r = Z/2$ = radius of pressure bulb of intensity ρ lb/sq. ft. = 1·57 ft.

Where Q = Surface-concentrated load

Z = Depth of the point

σ_z = Intensity of stress

$I\sigma$ = Influence value

$= 0{\cdot}4775$ for $\frac{r}{z} = 0{\cdot}00$

$\therefore$ Volume of soil contained in this pressure bulb $= 4/3\ \pi r^3 = 4/3 \times 3{\cdot}14 \times (1{\cdot}57)^3$

$= 16{\cdot}2$ cu. ft.

$\therefore$ Weight of soil participating $= 16{\cdot}2 \times 88 = 1425$ lb.

Step No. 2 :—

$\beta = 150000$ lb/cu. ft.

$$h = \frac{\text{Unit Static load}}{\rho} = \frac{1070}{\frac{\pi}{4} \times \left(\frac{19{\cdot}2}{12}\right)^2 \times \frac{1}{88}} = 6{\cdot}05\text{ft.}$$

$$r = \frac{a}{b} = 1$$

$$S = \frac{\alpha h}{b} = \frac{6{\cdot}05}{b} \quad \alpha = 1 \text{ (assumed)}$$

$$= \frac{6{\cdot}05}{(19{\cdot}2/12)} = 3{\cdot}78$$

Step No. 3 :— Refer to log-log Graph of Fig. 4.

$$\frac{r_z xy}{r} = 5{\cdot}2$$

$\therefore r_z xy = 5{\cdot}2$

$$\therefore k_z xy = \frac{\pi}{4}\beta d^2 r_z xy$$

$$= \frac{3{\cdot}14}{4} \times 150 \times 1000 \times \left(\frac{19{\cdot}2}{12}\right)^2 \times 5{\cdot}2$$

$$= 157 \times 10000 \text{ lb/ft}$$

$= k$ modified

Step No. 4 :—

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k \text{ mod } g}{W_v + W_s}}$$

$$= \frac{1}{2\pi}\sqrt{\frac{157 \times 10000 \times 32}{(1070 + 750) + 1425}}$$

$= 19{\cdot}8$ *C.P.S.*

Observed resonant frequency = 19·60 *C.P.S.*

Table 1. gives the natural frequencies calculated by the new method for the published results of Converse. It may be seen from this table that the results of the new method agree very closely with the experimentally determined results of Converse.

Similar calculations for the values given by Pauw were made and the results are shown in Table 2. Once again it may be observed that the predictions of the new method are in close agreement with the values obtained by Pauw both experimentally and analytically.

The new method was applied to several cases reported by Eastwood. Tables 3 and 3(a) show how the results of the new method are in agreement with those of Eastwood.

It must be pointed out at this stage that, while considering the dynamic load, the maximum positive value must be taken for the purpose of calculation. It has been observed by the authors in their experimental work using a Degebo-type vibrator that, although the dynamic load is withdrawn, the pressure within the soil does not instantaneously reduce to zero. Therefore, in spite of the dynamic load fluctuating sinusoidally in a Degebo type vibrator, the pressure within the soil will steadily increase up to a maximum and remains fairly constant thereafter. Hence it is necessary to consider the maximum positive value of the dynamic load in the calculations. For cases of free vibrations as in a foundation subjected to a single impulse (a hammer blow on a foundation block), the dynamic load may be taken to be zero, as the energy of a single impulse may

Table 2.—Application of the new method to the worked problem of Adrian Pauw.

Weight of Block	Dynamic load	β	Observed res. Freq.	Calculated Nat. Freq. (by the authors)	Remarks
3675 lb.	600	270,000 lb/cu.ft.	23·00	23·00	24·7 C.P.S. as Found by Pauw.
3675 lb.	300		23·75	23·75	

Table 3.—Application of the new method for the case of free Vibrations (The readings of Eastwood have been utilised)

Diam. of Base plate in.	Dead load in lb/sq.ft.	Assumed β in lb/cu.ft.	*f* nat. as found by expt.	Calculated *f* natural	Reference of graph	Remarks
6	1140	185000	45·00	43·40	Fig. 11	
6	700	185000	47·00	46·00	Fig. 11	
6	800	185000	47·50	46·50	Fig. 11	
6	1000	185000	44·00	44·00	Fig. 11	*
6	600	185000	52·00	48·50	Fig. 11	(a)
8	950	150000	35·00	34·40	Fig. 11	
8	760	150000	37·00	38·00	Fig. 11	
8	620	150000	38·00	38·20	Fig. 11	
8	380	150000	45·00	42·70	Fig. 11	(a)
8	1000	150000	34·00	34·20	Fig. 11	
8	800	150000	36·00	36·20	Fig. 11	*
8	600	150000	40·50	39·20	Fig. 11	
8	400	150000	49·00	42·00	Fig. 11	(a)
10	800	115000	27·50	29·50	Fig. 11	
10	600	115000	32·00	31·70	Fig. 11	*
10	760	115000	28·00	29·00	Fig. 11	
10	400	115000	37·50	34·40	Fig. 11	(a)

*This natural frequency is utilised to arrive at β value, (a) shows that for low loads the error may be considerable

Table 3 (a)

6 × 6	1000	125000	33·00	34·80	Fig. 12	
6 × 6	800	125000	38·00	36·80	Fig. 12	*
6 × 6	600	125000	45·00	38·90	Fig. 12	(a)
8 × 8	625	120000	32·00	33·60	Fig. 12	*
8 × 8	500	120000	37·00	35·50	Fig. 12	

*This natural frequency is utilised to arrive at approximate β value. (a) shows that for low loads the error may be considerable.

Note : All calculations are by slide rule.

just oscillate the soil contained in the static pressure bulb and does not appear to contribute to the increase in the volume of the soil participating in the vibrations. This concept is confirmed in the application of the new method to Eastwood's work where the experiments dealt only with single impulses.

6. Conclusions

1. The method suggested is very simple as it involves only the determination of the mass of soil contained in a pressure bulb corresponding to ρ lb/sq. ft. where ρ is the value of the density of soil in lb/cu. ft.
2. This method for predicting the natural frequency of a foundation soil system is valid for either free or forced vibrations.
3. Calculations should include the value of maximum positive dynamic load in the case of forced vibrations and no dynamic load in the case of free vibrations.
4. This method is valid for single degree of freedom.
5. The validity of this method for cases of foundations below ground level or supported on springs is not verified for want of data for such conditions. The authors are engaged in experimental investigations in these fields.
6. The authors feel that this method may not give correct results for foundations transmitting low intensities of loads.
7. This work confirms the earlier observation that the natural frequency falls with the increase in dynamic load. It is because of the fact that maximum positive dynamic load and the increased weight of soil, due to the dynamic load are terms in the denominator in equation 1, which bring down the natural frequency.

7. Acknowledgements

The authors wish to place on record their indebtedness to Professor N. S. Govinda Rao, for his unceasing interest in this investigation. They express their thanks to the authorities of the Indian Institute of Science, The Central Board of Irrigation and Power, and the Council of Scientific and Industrial Research (Govt. of India), for the facilities afforded for the investigations.

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Appendix. 1

The modulus of elasticity of soils is a conflicting topic as the stress-strain curve for soils is throughout a non-linear one. The subject becomes all the more complicated when we consider its variation with respect to depth.

Prof. Christopherson assumes that the modulus of elasticity has a finite value of E_0 at the surface and $2E_0$ at a depth x, $3E_0$ at a depth $2x$ and so on. Eastwood does not agree with this and states that this assumption is unlikely to be correct except in a thick bed of consolidated clay. The assumption made by Pauw again does not agree with this because Pauw assumes that the modulus of elasticity varies with effective depth in the case of non-cohesive soils and is a constant in the case of cohesive soils.

However, here the authors assume that Pauw's assumption is correct for the purposes of this paper.

Appendix. 2

In the equations for the Spring Constant, a constant β is involved. It appears at first sight that this constant depends on the area, the effective depth etc. As sufficient information about its variation with these factors is not fully known, the authors suggest modifying the equation 5(b) to $K_z{}^{xy} = \lambda b^2 r_2 a^{xy}$ where λ is constant taking account of these unknown factors, thus maintaining the form of the equation. In the present work the authors have considered that $\beta = \lambda$ and assumed α as 1, for want of enough data.

Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by December 31st.

Book Reviews

F.B.I. Register of British Manufacturers, 1960. 32nd Edition. Published January 1960, for the Federation of British Industries. (London: Kelly's Directories Ltd. and Iliffe and Sons, 1960). 9½ in. × 7½ in., 1164 pp. 42s.

This F.B.I. Register, is a comprehensive guide to a substantial cross section of British Industry, containing lists of the products and services of over 7,500 member firms under more than 5,400 alphabetical headings. In addition to the Classified Buyers' Guide there are seven other sections in the Register, giving addresses of Companies and Firms, information about Trade Associations, proprietary names, trade marks, etc., and French, German and Spanish glossaries.

Concrete Formwork Designer's Handbook, by H. R. Gill. (London: Concrete Publications, Ltd., 1960). 9¼ in. × 6 in., 153 + vi pp. 15s.

This book gives more than fifty nomograms, tables and other charts for the rapid design of the components of the formwork, or shuttering, for vertical or inclined walls, straight or curved walls, beam-and-slab construction, and dams, and continuously-moving forms. The various factors affecting the lateral pressure of freshly-placed concrete against the forms are evaluated and the stresses permissible in timber, plywood, and steel under various conditions are given. The design aids are supported by the basic formulae from which they are derived, but the derivations of the formulae are given in an appendix so that the practical parts of the book are free of theoretical analysis. Several fully-worked examples explain the application of the aids.

Practical Prestressed Concrete, by H. Kent Preston. (New York and London: McGraw-Hill, 1960). 9 in × 6 in., 89s.

This book is a practical design manual, intended for use by designers working to current American rules. It sets out in full the Tentative Recommendations for Prestressed Concrete prepared by a joint committee of the American Concrete Institute and of the American Society of Civil Engineers, and half the book is devoted to showing by practical example how the design formulae are used. It gives an adequate background of information on constructional technique. The book is well planned, and except for the inconvenient method of cross-reference, easy to read.

The Author has concerned himself with only the detailed and practical application of the Recommendations, and not with any consideration of their derivation or validity. A substantial bibliography has been provided, however, for those who wish to probe below the surface. F.N.P.

The Design and Construction of the New Assembly Building for the Ford Motor Company Limited, Dagenham*

Discussion on the Paper by D. Lax, M.I.Struct.E.,

and

F. T. Bunclark, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E.

The Chairman introduced the authors who then presented their paper and showed a film illustrating the progress of the work on the foundations and the structure of the Assembly Building at Dagenham.

MR. LAX in his opening remarks, emphasized that a project of such magnitude was the work of a large team.

Mr. Martin Hutchinson was the Architect associated with the Consultants for the design of the facilities block and the external elevations to the main building. Mr. Arthur J. Norcliffe co-operated with him on the ancillary buildings on the south side.

The method used for dealing with the differential movement between the external brick walls and the structural frame to restrict the cracking in the brickwork was a problem often experienced in dealing with a large expanse of brickwork.

Full height vertical joints were incorporated every 45 ft. by the construction shown in Fig. 24 with the inner leaf held to the secondary steel framing posts every 11 ft. 3 ins.

The method of forming the main expansion joints was also illustrated in the Figure.

In addition to the normal superimposed loading on the structural steelwork, all main and secondary members (excluding filler joists) were designed to carry a non-accumulative point load of three tons in any position to permit the suspension of plant from the structure, the details of which were unknown at the time of design. At first appraisal this would appear to be conservative, but Fig. 25 was typical of the conditions to be provided for in a modern plant building where a considerable part of the equipment and services are overhead.

It had been found that bitumen felted roofs laid on cement screed over light-weight insulated slabs were prone to splitting of the felt through all its plies across a full section of a roof or monitor, generally after a period of warm sunny weather following the shutting-off of the heating at the end of the winter.

The reason for this appeared to be due to the change of humidity in the screed on the cold side of the roofing caused by the early summer sun, with a resultant rapid shrinkage of the screed.

**Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1., on the 14th January, 1960. Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E.(Vice-President) in the Chair. Published in "The Structural Engineer" Vol. XXXVIII, No. 1, pp. 1-19.*

The galvanised wire reinforcement incorporated into the screed of the light-weight roofing at the Ford Motor Company restricted the extent of the splitting, but did not prevent it. Where long lengths of roofing were laid continuous, it had been found advantageous to incorporate the type of joint shown in Fig. 26. The roof was laid as described in the text, but before screeding a timber batten was laid over every tenth joint of the wood-wool panels, generally twenty feet centres.

After screeding the batten was lifted and the wire cut through on the joint which was then filled with mastic. A strip of hessian based felt was laid over the section, bonded at the outer two inches only on each edge. The bituminous felt was then laid in the normal manner.

That type of joint permitted a greater length of movement of the felt in contact with the screed, thus preventing excessive tension in the felt and it also ensured that the movement occurred at defined positions.

MR. BUNCLARK, describing the site construction said the foundations and ground floor were straightforward civil engineering work with some special methods adopted and accurate programming to be ready for the superstructure commencement.

In the superstructure contract some 40 specialist sub-contractors were employed under the main contract, followed by some 20 major direct contracts for the supply and erection of the manufacturing plant to be installed in the building, all requiring their particular floor areas to be free for their entry as early as possible, so that the programming and dovetailing in became considerably more intricate

At the peak nearly 2,000 men were employed on the site which needed an organisation of its own in providing canteen, first-aid, welfare and safety facilities

THE CHAIRMAN proposed a vote of thanks to the authors for their admirable paper and their equally admirable presentation of it and the film. He hoped that a lot of the additional information the authors had given in presenting it would be published in the Journal.

MR. R. C. W. BEDFORD (Associate-Member), was particularly interested in the paper because he worked next door to the Ford Company's plant alongside the river and the foundation problems there were of the same sort as those which had confronted the authors.

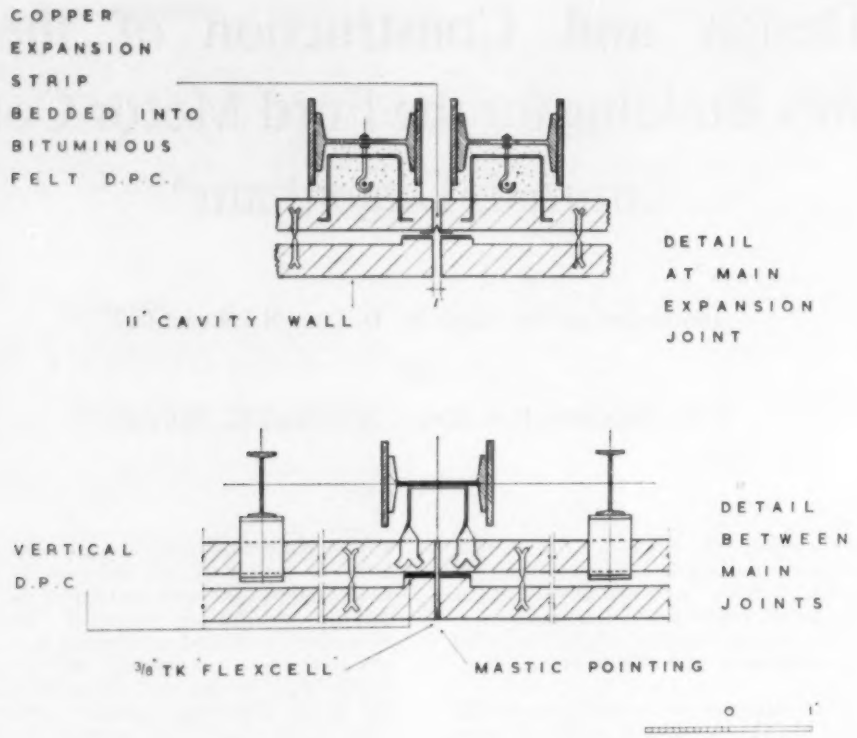

COPPER
EXPANSION
STRIP
BEDDED INTO
BITUMINOUS
FELT D.P.C.
11" CAVITY WALL
DETAIL
AT MAIN
EXPANSION
JOINT
VERTICAL
D.P.C
3/8" TK "FLEXCELL"
MASTIC POINTING
DETAIL
BETWEEN
MAIN
JOINTS
0
1'

Fig. 24

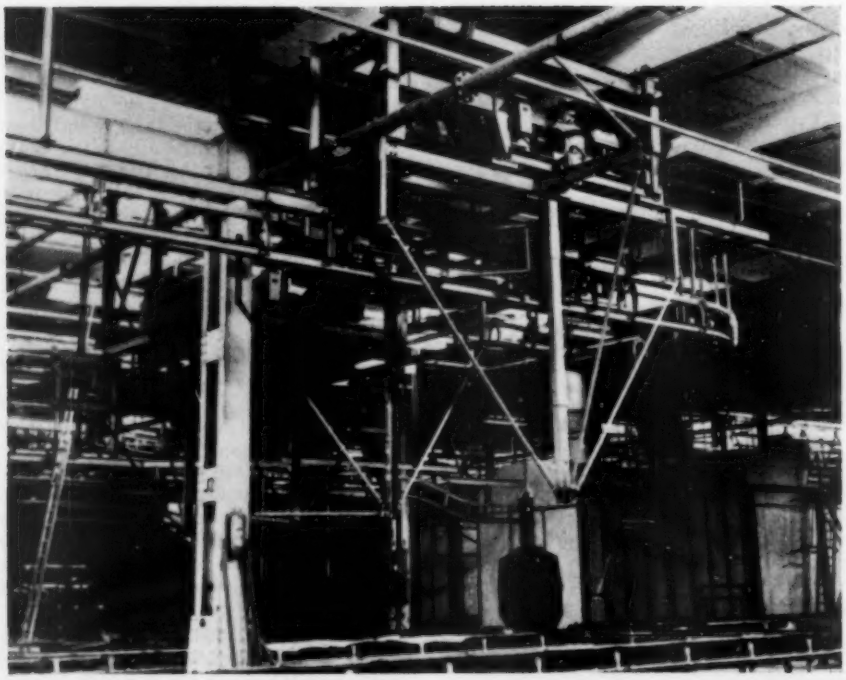


Fig. 25

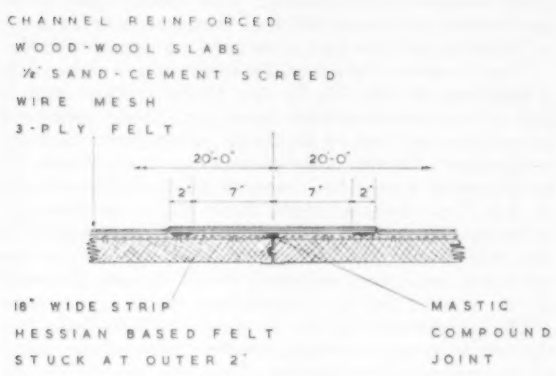


Fig. 26

The piling had interested him particularly and he was not clear as to whether the piles were driven through the peat to the clay and ballast or driven as far as they would go, as in the case of the test piles. At the site where he was working they tried to stop at the underside of the peat and put a load of at least 55 tons. He wondered if this was safe.

He asked what advantage was gained in prestressing by using prestressed piles.

The only foundation failures experienced at his site were with bored piles. He asked whether precautions were taken at the Ford Company's site against the enormous amount of sulphates in the peat.

He was interested in the construction of the drying oven floors and asked the reason for the elaborate construction adopted.

Finally he asked the reason for the expensive 'ship-lap' construction used on the conveyor bridge.

MR. T. BEDFORD (Member), said there were many aspects of this very large building, both as regards the design and construction; questions also arose on the performance of the building after the occupation of it by the clients.

In regard to design he said the steelwork appeared to be unprotected. The Model By-laws required that floors and stanchions in buildings should be protected against fire, but the designers of this building had managed to avoid the fire protection which normally one was expected to provide, at least an hour's protection. In the paper he did not see any reference to it whatever.

In respect of floor loadings, one could appreciate very well the designers' difficulties in assessing them. The authors had said that the loading figures were 300 lb. on the first floor and 3 cwts. on the ground floor, and he asked if they could say how near their assessments of loading were to the actual conditions when the factory was in operation. He asked the question because those who had had anything to do with British Standard Specifications realised the immense amount of work that was done in bringing the specifications up-to-date and the discussions which took place in order to get an extra half a ton stress allowance; yet when we came to a building of this nature we were faced with a real problem in regard to loading. He did not mention this as a criticism; it was a difficulty which the designer had to face in arriving at the true loading of a building of this nature.

He was very interested in the technique adopted for the floor construction. The ground floor slab was laid 15 ft. wide and 235 ft. long continuously with dummy joints at 20 ft. centres. He asked whether there had been any movement in the slab; had they observed movement at the contraction or expansion joints?

The floor finish presented one of the principal and most important sources of trouble in a factory, and in this case granolithic was used. One would naturally expect that cracks would appear at the joints; in fact, that was why the designers had put the dummy joints there. He would have thought that in addition cracks would occur between those joints. He asked what preparations were made for dealing with the cracks when they appeared, because they presented one of the most serious troubles in the life of a factory floor.

The same sort of question arose in connection with the first floor, where there was filler joist construction. Again he would have thought that cracks would occur in the granolithic floor over the tops of the filler joists, and he asked whether they did in fact occur.

A problem for the consulting engineer who was faced with a contract of this kind was how to go about the arranging of the contract in order to meet the time schedule. He asked whether in this case the consulting engineers negotiated the contract, or was it by tender, or what method was adopted on the financial side for running this very big contract.

MR. M. R. BROOME (Associate-Member), speaking of the suspended slab on the first floor, said the great point was the method of shuttering. He referred to the cover at the bottom of the filler joists and asked whether the authors were happy about it from the fire hazard point of view and whether they would use it in other buildings.

Next, he asked if they would comment on the number of test piles used and would say whether the information acquired by their use was worth the trouble, bearing in mind the wide range of pile lengths used on the site.

Speaking of the construction of the ground floor slab, he said the method adopted was not normally used for a reinforced concrete slab of this nature. He asked whether it was really applicable to that purpose and whether there were any defects in it.

MR. J. M. D. GURNEY (Member), asked whether any special calculations were necessary in choosing the length of about 300 ft. between expansion joints. A problem in connection with a factory of this kind arose from the extreme differences of temperature and presumably of humidity, and he wondered if that had affected the authors' design in respect of the distance between expansion joints.

MR. G. S. MURRAY (Graduate), asked if the authors considered that the rake of piling used was absolutely essential in dealing with the site conditions existing at Dagenham.

MR. A. J. TAYLOR (Ford Motor Co. Ltd.), correcting a misunderstanding, said the floor loading was *given* to the consultants.

MR. T. BEDFORD pointed out that he had referred to the 'assessment' of loading.

MR. GEORGE C. WRIGHT (Member), said he had noticed in the film that two men handled slabs 11 ft. long and 2 ft. wide. Were there any complaints regarding the weight being so handled?

A further question was why it was decided to make the span 11 ft. 3 in. What was the permanent deflection after the dead load had been applied? And had any puddles been noticed since it was finished? Being

the original designer of that system of decking he and his colleagues had confined the lengths they used to 10 ft. because of the permanent deflection.

MR. M. R. BROOME said that from the film it seemed that there were innumerable vents all over the place. He asked if they were really necessary in such large numbers and whether there would not have been a better architectural way of dealing with them.

LT.-COLONEL G. W. KIRKLAND, M.B.E.(Mil.) (Vice-President), said he had noted from the film that almost at the stage of the completion of the main building there was a deep excavation being formed, and he wondered could they have the reason. Also it was not clear whether the sheet piling was driven simultaneously.

Recalling the remarks of Mr. T. Bedford, he said it was on rare occasions that we heard the word 'contraction' used in respect of the joint which were provided to take care of thermal movement. In the paper they were referred to as expansion joints, and he urged that engineers must be very very careful about the education of the young engineer, for it was very easy to regard a joint as being provided for expansion whereas more frequently it should be for contraction. He would like to have that point on record.

He had not found any indication in the paper that use had been made of Universal Beams, provided they were available at the time the building was being constructed; he invited the authors' comment.

THE CHAIRMAN, commenting that most structural engineers had to do with piling, asked if a little more information could be given concerning the test loads and settlement diagrams.

He had wondered whether a 2-ton hammer was rather light for a pile which weighed something approaching 4 tons. He had also wondered whether the 4 ft. drop was too much and whether it might tend to overstress the heads of the piles. Were there any marked effects on the heads of the piles due to driving stresses? In the same connection, what packings were used in the helmets under the diaphragms? He asked whether any temporary compressions were measured at any stage.

Again, he would like to know whether the piles were driven into the sand or into the clay material shown in Fig. 6. They could have been driven into the yellow sandy clay or into the grey sandy clay or into the grey sand. From a rough calculation it seemed that if the toes rested in the sand the toe load was about 100 tons and the friction about 50, whereas if the toes rested in the clay material the toe load would be only about 70 tons and the friction much about the same. He suggested that there was a marked difference between a pile in sand and one in clay.

As to the reason why prestressed piles were used, he asked if they were cheaper than others or more readily available. He was glad to see that their size was 14 in. × 14 in., which was a satisfactory cross-section, and he asked what was the strength of the concrete used in the piles.

MR. GEORGE C. WRIGHT said that on some of the jobs he had designed he had a great deal of trouble with paint. It seemed to him that when paint was applied at the fabricating works they overloaded it with T.V.O., a tractor vaporising oil or turps substitute. When steel was exposed to wind or sun the paint oxidised and faded away into dust and it washed off with the first rain. He had noticed that, in the course of the construction of the new assembly building at Dagenham, the paint was applied after the steel was delivered to the site in the rolled condition. Perhaps it was done under the supervision of the Ford Company's engineers, who would not allow turps substitute or anything of that sort to be used.

Could the authors give some enlightenment on that?

THE AUTHORS replied to the discussion:—

Replying to Mr. R. C. W. Bedford, they stated that the general strata found in that area led to a wide variation of depth for piling, even on relatively short distances between piles. In one case on an adjoining site it had been found that piles driven at 6 ft. 8 in. centres would drive fairly consistently to between 20 to 25 ft. penetration until quite suddenly one pile would be literally 'lost' and had to be extended sometimes as much as 10 ft., yet the next in line would bring up at its normal depth.

There appeared to be a stable ballast bed over the whole area at variable depth, and in the opinion of the authors the sudden variation in pile length was caused by water courses cut into the ballast generally in a north-south direction normal to the River Thames.

Wherever possible the piles were driven a foot or two into the ballast, but not through it, but the overriding condition was a minimum set of 2 inches per 10 blows on three consecutive sets. On the majority of piles driven on the site it was found the final set was of the order of ¾ inch per 10 blows.

The strata and penetration indicated in Fig. 6 was only typical to show the wide range of variation at different parts of the site between the two diagrams shown.

As an indication of the range of piling variation the following table giving the length of piles used might be of interest:—

Pile Length in feet	Percentage of Total Number of Piles used
20 to 25	20
25 to 30	50
30 to 35	28
odd	2

The safe loading on the piles had been assessed in the normal way from the test results, and it was also necessary to determine what was a reasonable working load for pile groups as against single piles to meet the conditions applying to the building.

No distinct technical advantage was gained by the use of prestressed concrete piles—it was just a matter of supply and demand. Prestressed concrete piles were available in the quantities required at an economic price, whereas there was a delay and a limited supply of normal reinforced concrete piles. Stringent tests had previously been carried out on prestressed concrete piles and found satisfactory. The concrete specified strength was 6,000 lb/sq. in. at 28 days—70 per cent of this strength was achieved in 24 hours by steam curing, for lifting from beds.

As stated in the text, sulphate resisting cement was used for all in-situ concrete below ground level, including in-situ piles; the latter were only used where they had to be placed under overhead power transmission lines with insufficient headroom for a pile frame.

As stated in the text, the oven floors were subjected to high temperatures, and the method described was used to isolate the oven floor from the main structural floor and to provide insulation.

The authors could not agree that the method of cladding the bridge was expensive—in fact it had proved remarkably quick and economical, bearing in mind the temperature and humidity conditions that had to be met, together with good architectural appearance.

In answer to Mr. T. Bedford, the question of fire protection for this building was discussed with the Local Authority, with particular reference to the encasement of the steel frame.

Under Section 63 of the Public Health Act 1936, there was a provision for relaxation of the requirements of a Bye-law where the Local Authority considered that the operation of any building Bye-law would be unreasonable in a particular case. In this case the view was held that, having regard to the siting of the building, the provisions made for fire fighting, alarm and means of escape, together with the purpose for which the building was to be used, it would be unreasonable to insist upon compliance with the full standards of the Bye-laws of fire resistance for the structure.

Regarding the assessment of floorloading, as Mr. Taylor had said, the Consultants in this case were given the loading for which the floors were to be designed; the authors were sure that this was based on experience in America with a similar type of plant. They were sympathetic to the view expressed by Mr. Bedford, for in a number of cases with which they had to deal the loading variation far outweighed the limits imposed by normal methods of calculation.

As to the movement of the ground floor slab at the expansion joint, the authors assumed that Mr. Bedford's first question meant movement at the main expansion joint of the structural slab. There were no dummy joints in this slab.

There was every indication that the building had moved at these joints, but the amount of movement was rather difficult to record. Perhaps Mr. E. Kimber, who was the Resident Engineer on the site for the work, could say something on this point.

For the first floor, movements of up to ½ inch had been recorded, which indicated that the joints were operative.

Having experienced trouble with floor finishes on various other contracts, the Consultants were very concerned about this, and discussed the matter at some length with the Client.

The floor finish on the ground floor, which was finally agreed on and in which dummy joints had been incorporated, had over the major part of the floor area behaved well, there being no obvious cracking or complaints concerning the finish of the floor, but in one particular section trouble had been experienced, the cause for which could be attributed to a number of reasons. There appeared to be little one could do when lifting or excessive cracking occurred on floor topping, except hack up and relay.

Their work was organised in four separate contracts drawn up as information became available from the Client.

The first was negotiated to enable an early start to be made on the site—the remainder were by tender.

All contracts were placed on Bills of Quantities for a type contract and re-measured. The following were the main headings under which the Contracts and Bills were prepared :—

Contract No. 1.—*Site Works*
Bill No. 1. General & Preliminaries.
2. Site Clearance & Levelling.
3. Roads, hardstandings, soiling and sowing.
4. Drains (external to the building).
5. Services.
6. Test boring and piling.
7. Daywork.

Contract No. 2.—*Piling*
Bill No. 1. General & Preliminaries.
2. Piling, including cut-off to standard level.
3. Daywork.

Contract No. 3.—*All construction up to finished ground floor level*
Bill No. 1. General & Preliminaries.
2. Foundations & Floor Slab
3. Drains & Services under Building.
4. Daywork.

Contract No. 4.—*Superstructure, including specialist services*
Bill No. 1. General & Preliminaries.
2. Prime Costs & Provisional Sums (generally for specialist services).
3. Superstructure—Main Building.
4. Ancillary Buildings (complete).
5. Coupling of drainage and other services, boundary fences, etc.
6. Daywork.

MR. E. KIMBER confirmed the statement made by the authors regarding the movement of the main structural slab on the ground floor.

There had been some cracking on the first floor, but it was not excessive. Cracks had occurred over filler joists, but only in very limited areas.

Bearing in mind the reply already given to Mr. Bedford on the subject of encasement of steelwork, the authors commented there was little that could be said in reply to Mr. M. R. Broome's question. The form of construction adopted had enabled an exceptional speed of construction for the floor and the small suspended shuttering panels had made this possible.

In regard to hazard due to the exposure of underside of a steel filler joist when the remainder of the section was cased; this had been a common form of construction over the years, and the author had never seen or had knowledge of a filler joist floor failing by the exposure of the underside of the joist due to fire.

As to the number of test piles used, the authors had tried to indicate in the text that it would have been preferable to have driven a lot more test piles. It had only been possible to provide a pattern of test piling from which an assessment of the length of piles required in various parts of the site could be made. Whilst this had not proved entirely successful, the information gained from the driving of test piles had been most useful, and of course, these piles were incorporated into the work.

To complete the construction of the ground floor slab in the programme time, something abnormal had to be done. The use of road machinery proved very satisfactory, both in exceptional rate of progress and quality of work. The even spacing of reinforcement over the floor allowed the forms to be slotted at even centres and used consecutively on the large standard areas.

In reply to Mr. Gurney's question very little real data existed on the length that should be adopted between

expansion joints, and it generally seemed to be resolved by practice and engineering opinion. There had been discussions between the Consultants and the Ford Motor Company's Engineers in this country and from America. The American practice was to have much greater lengths between joints than is generally used in this country, and the length used for this building was a working compromise based on the siting of the plant.

The slab was heavily reinforced compared with road and airfield slabs. The latter are now being constructed in lengths up to 400 ft. without a contraction joint.

The reference to the extreme difference of temperature and humidity was presumed to refer to the drying ovens and the wet sand decks. These were, as far as possible, isolated from the structural floor, but of course, they must cause an overall rise in temperature and humidity in the building, which has to be countered by the ventilation plant. So far, no major adverse effect had been apparent on the main structural frame or floor from this cause.

Replying to Mr. G. S. Murray, the authors stated that the raking of piles had no relation to the site conditions. It had been done solely to restrict the size of the group pile caps under the main stanchions and ensure reasonable distribution to the bearing strata. If driving had continued vertically at 4 ft. 6 in. centres for large groups of piles the cap size would have been very large and costly.

The Consultants had recommended their Client to use wood-wool roofing slabs where applicable because of advantages of weight and insulation. One of the authors was interested in Mr. Wright's reference to the development of the interlocking steel edged channel slabs. He himself had also been concerned with the development of continuous wire reinforcement in the screed as he had found this considerably stiffened the slabs when formed into a complete roof structure. It was from information gained in the testing of this form of roof that the Consultants were satisfied that the span/deflection ratio on 11 ft. 3 in. span was quite permissible for this form of roofing. The maximum weight of the wood-wool panels used was 200 lb. handled by two men, as shown in Fig. 13 in the Paper. It had been the Consultant's practice for some time to use pressurised overhead collection pipes taken to the outside walls before discharging to the surface water drains. The collection of water from the roof could also be assisted by making use of the natural deflection of the structure by siting the collection heads at the centre of the span, but this, for other reasons, was not always possible. The means for avoiding excessive pooling of water in the building under discussion was described in the text.

In reply to Mr. Broome's question on ventilators, it was necessary for the ventilator chimneys to be sited directly over the plant being exhausted, for grouping was discarded on the grounds of safety and economy, both of space and cost.

The authors knew of no attractive method of dealing with this problem, and if there had been, they were sure that Mr. Martin Hutchinson, who was the Architect advising the Consultants, would have suggested it.

Replying to Colonel Kirkland, the authors stated that the excavation in question was for the tank of the paint separator described on page 13 of the text. The steel sheet piling for this was driven after the structure for the main building had reached an advanced state of completion. Construction of this type had been carried out previously at other parts of the Ford Motor Company's site, all of which were adjoining structures with concrete piled foundations. No trouble had been experienced.

The authors appreciated that the term 'expansion joint' was a misnomer in that there was contraction as well as expansion occurring. Providing the function of such a joint was generally understood, was the name so important?

Concerning the use of Universal Beams, it has been stated in the text that the Consultants and Contractors were prepared to consider the acceptance of any steel of a suitable specification. If Universal Beams had been available they would have been used.

In answer to the Chairman's remarks, the authors commented that when dealing with a paper of the type presented, it was difficult to know what could be pruned to within readable limits. The term 'test pile' used in the text covered all piles driven initially to prove the strata in conjunction with the bore records. Of these piles only four were load tested, and the following data covers the tests; these test piles were sited in areas where the proving piles had indicated that further information on acceptable sets were necessary.

Pile Position	A/28	B/17	F/17	N/7
Depth. Penetration (Feet).	21·25	24·0	25·0	23·75
Set (Inches).	1·875	2·0	2·0	2·0
Load (Tons).	90	87	91	91
Deflection (Inches) Initial	0·20	0·19	0·22	0·19
Ditto. After Seven Days.	0·20	0·19	0·22	0·19
Recovery (Inches) Elastic.	0·12 Y	0·12	0·08	0·07
Permanent Deflection	0·08	0·07	0·14	0·12

No head damage was experienced on piles due to driving stresses—1 inch plywood packing with a plastic dolly in the helmet was found to be the most satisfactory method in driving.

In reply to Mr. G. C. Wright, the normal standard of workmanship in cleaning and painting of steelwork at the fabrication works often left much to be desired. It was not unknown for paint to be applied from a bucket with a tarbrush over dirt, scale and even swarf from drilling. Whilst there had been a realisation of the need for improvement, and some firms were offering shot blasting and metal spray facilities, in general the Consultants preferred to have all steel erected unpainted except for contact surfaces.

The Ford Motor Company specified the painting procedure, but this was supervised by the site staff of the Consultants.

Folded Plate Construction: An Investigation of Collapse Conditions*

Discussion on the Paper by A. R. Dykes, B.Sc., A.M.I.Struct.E., A.M.I.C.E., A.R.C.S.T.

THE CHAIRMAN, proposing a vote of thanks to the author said he was glad to hear him say that he had treated his subject simply. The essence of good design was simplicity, but one felt that that was sometimes overlooked. One was glad that the subject could be reduced to simple terms, and it was a pleasure to propose a very hearty vote of thanks to him for having presented some difficult material in a simple way.

THE CHAIRMAN added that no good work was ever lost, and the more that engineers would put on record even the simplest of their observations, made either in the laboratory or in the field, the better it would be for the engineering profession as a whole. Maybe in the future someone else would present a paper in which reference would be made to Mr. Dykes' work.

MR. S. ROSENAK (Associate-Member), congratulated Mr. Dykes on his very interesting paper and asked for his comments on two matters.

First he referred to the load/deflection curves for slab elements (Fig. 8) and said that in a slide which the author had used to illustrate them there were two discontinuities in the curves for structures IB and IIIB. These occurred at the appearance of the two yield lines, at the ridge and within the slab. Looking at Fig. 8, he wondered whether the curves for structures IB and IIIB should not be more accurately drawn to show these discontinuities. It seemed to him that there would be two points of increased deflection in the slab when the two cracks occurred. Regarding the support conditions, structures IB and IIIB were supported vertically and horizontally along the longitudinal edges. In practice one might have folded slabs where the edges were supported vertically or horizontally only and free to move in one direction.

Referring to some hipped plate structures which had been illustrated, he asked if it would not be more realistic to load the tops only and not the sides. In the author's tests the tops and sides were loaded normally with equal loads. In practice the loads on the relatively steep sides would be small compared with normal loads on the tops and he invited the author's comments on this loading.

Mr. HUGH FRASER (Member) said it was a pleasure to attend the meeting to hear a fellow member of the Scottish Branch who had the honour to give a paper to the Institution at headquarters.

Having noted from the diagrams that the forces were all normal to the surfaces, Mr. Fraser commented that the important factor about any structure was that of the gravitation forces.

Another practical point arising was that there was a very rigid support for the two sides and the horizontal reactions were acting. He asked if these were affected by the author's arrangement of loading normal to the rafters.

Mr. H. WORRELL (Associate-Member), added his thanks to Mr. Dykes for his interesting paper.

He was interested particularly in the method of loading in the tests described, and he asked if Mr. Dykes would be good enough to outline how a symmetrical loading could operate on such a structure in practice. If the loading on the models was unsymmetrical he would be interested to know whether Mr. Dykes expected a similar pattern of collapse to apply.

Mr. A. L. PERCY (Member of Council), said he was very interested to read the paper and to note the close agreement between the experimental results and the theory which the author had used.

Not being familiar with the Fracture Line method, he had looked up the work of Johansen and had found it stated that the basis of his method was that elastic deformations were very small in comparison with plastic deformations. That being so, in plain sections the fracture lines should be straight. But Mr. Percy had noticed that quite a number of the fracture lines shown in the diagrams in the paper were curved, particularly in Fig. 5c some were very curved indeed, and he would be interested to hear Mr. Dykes' comments on that.

He felt that this method might be useful for the design of some small roofs he had in mind, and he asked to what extent the results could be magnified and applied to practical size structures.

Mr. J. D. BENNETT asked Mr. Dykes about the support conditions at the ends of the plates. From what he had seen of the diagrams in the paper and the slides which had been shown, all the end reactions had to be normal to the planes of the plates, since the plates merely rested on the end supports. If this was so then it would be impossible to develop any appreciable shear forces in the plane of the plate, consequently no longitudinal forces would be developed in plate and this would explain why no large tensile cracks occurred at midspan of the folded plates. Mr. Bennett thought that the principle of the shear forces being mainly in the plane of a shell or folded plate was one of the main advantages of using those types of structures, and for the shear to be carried in this manner it was essential for the folded plate to be built in to stiffening beams or frames at the ends.

It appeared from the paper that all the tests were more or less on continuous slabs and not on "folded plate structures" in the accepted sense of the term since the loading and support conditions were such that the strengths of these models depended only on the slabs resistance to bending in directions

Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1., on the 25th February, 1960. Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (Vice-President) in the Chair. Published in "The Structural Engineer" Vol. XXXVIII, No. 2, pp. 60-67.

perpendicular to its plane and no appreciable direct forces could be present in these structures.

Mr. ROSENAK, referring again to Figs. 7 and 8, said that structures IB and IIIB were rather similar as regards load and deflection, they were both of a type which gave rather similar load/deflection curves. One also noted from Table A that the collapse loads for those two structures were rather similar. From the practical point of view, if the author's support conditions were realized, he wondered if it was worth while to support the slab at the transverse edges, as in type IIIB, as type IB would give a structure of similar strength to that of type IIIB. He invited the author's comments on this point.

Mr. I. L. DAVIES said that for a long time his organisation had been doing a lot of work on similar kinds of structures, mainly with a plain slab, very simply supported on the edges. When they applied the Johansen yield line theory to the analysis of those structures they found that their loads were considerably higher. He asked if Mr. Dykes could suggest a reason why, when the slabs were so simply supported, the Johansen theory did not apply.

Also, in work done by other organisations, mainly the Building Research Station, a loading on a very light slab had been obtained which was ten times the Johansen figure.

Mr. A. J. ASHDOWN (Associate-Member), said that Professor Ockleston had the opportunity to test a reinforced concrete building only ten years old. In applying Johansen's theory and also the latest plastic theory to a reinforced concrete two way continuous slab, the figures would come only within 35 per cent of the actual failing load.

Mr. W. H. AUBREY (Associate-Member) asked, in view of the difference between experimental results carried out at different establishments, whether the span/depth ratio was possibly the criterion.

THE CHAIRMAN emphasised that possibly the fixing of the end conditions was the factor in many of the results, and he asked if the author would comment on that again.

He was not sure whether the author had said that the cracks which had occurred were due to compression or to tension, but it had seemed to him that they might very well be due to tension.

Further, he recalled that some years ago a factory was built in east London in which just this form was used, but in steel frames which were laid against each other. He did not know whether this theory could be applied to the more difficult subject of the paper.

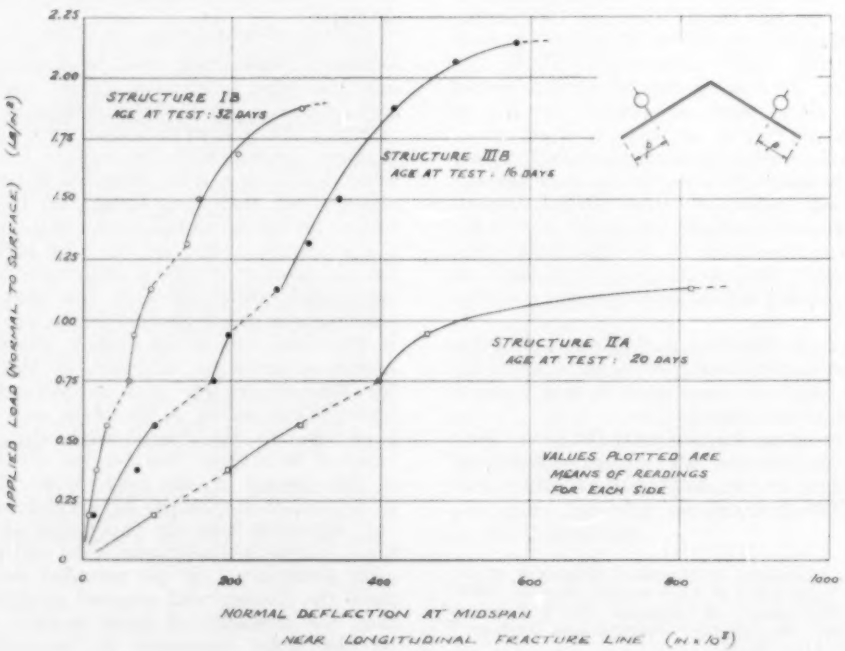
Mr. DYKES, replying to the discussion, said that in view of some of the remarks made by speakers it might be of benefit if, before answering questions in detail, he made some comments of a general nature.

His theme was simplicity and though too much simplification could be dangerous, it seemed to him that the time had come in this field (as in many others) to call a halt to the trend which carried design methods, without the assurance of more satisfactory structures, beyond the comprehension of many engineers. Methods of design which kept the structural problem before the designer surely had greater merit, provided, of course, that the end result was valid.

He wished to take his stand on the side of those who advocate design methods deriving from observation of structural behaviour rather than from highly theoretical studies alone.

Thoughts along these lines led to the study of the mode of failure of some folded plate structures and the possibility of their being considered as assemblies of slabs rather than as slab-and-plate elements. Work on the formulation of a collapse design method, based on the fracture line theory and providing satisfactory structures at least in some cases, was showing some promise, but much remained to be done. The work described in the paper and in his remarks while introducing it constituted only the basic idea from which it was hoped something of real importance would emerge.

In reply to Mr. Rosenak's remarks with regard to Fig. 8, he said that the slide referred to (and reproduced



below) had been shown for the purpose of illustrating the point made by Mr. Rosenak. It was in fact Fig. 8 redrawn to show the two discontinuities which had not been noticed when the graph was originally prepared. All subsequent tests had exhibited this feature.

As had been pointed out in the paper, it was not possible to compare quantitatively the collapse loads or deflections of Structures I and III because of the difference in their ages at test, and so Mr. Rosenak's suggestion that traverse supports were not worth while was not sound. Structure Type I was merely a cranked slab, one stage removed from a simple slab, and for a true uniform load there would be no longitudinal bending.

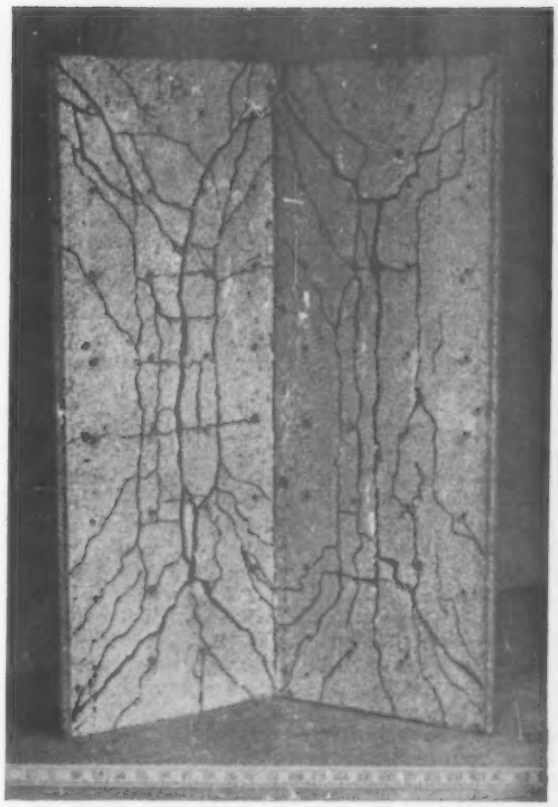
Mr. Fraser's remarks supplemented Mr. Rosenak's queries regarding the loading applied and the edge supports of the structures. In every case the loading had been applied normal to the surface of the slab elements forming the structure. The principal reason for this was experimental expediency; it was a type of loading which could be applied fairly easily and whose magnitude was known with certainty. As justification for its adoption he pointed out that for a typical folded plate roof comprising two slab elements, the normal component of a gravitational load would be at least 80-90 per cent of the vertical load. The component in the plane of the slab element would be in the region of 20-50 per cent, and varying from zero at the ridge to a maximum at the edge. Clearly the normal component was of major importance here. Referring to the structures with three slab elements, described while he was introducing his paper, he agreed that it was difficult to visualise the circumstances where there was the same normal load on the top as on the steep sides—but there were folded plate structures other than roofs where hydrostatic or pneumatic pressures might be encountered. However it was desirable in preliminary stages of an experimental investigation to eliminate as many variables as was reasonably possible and it was for this reason as well as to simplify the apparatus that normal loading was adopted.

The suggestion was made with respect to Structures I and III that the supports were not practical in that they allowed no translation either vertically or horizontally. He disagreed with this reference to practicality as he could readily visualise the case where no edge movement occurred, but he also admitted that frequently the supporting structure would permit some displacement. The complete horizontal restraint was of course recognised in the analysis.

In reply to Mr. Worrell's query about symmetrical loading in practice, he said that, while it might be far from the truth, it did represent a commonly adopted design condition in shell roofs and folded plate structures of symmetrical cross-section where, generally, only gravitational loads—self weight, snow and imposed load—were considered. If unsymmetrical loading did operate, the form of collapse would depend on the degree of asymmetry. If this were considerable then collapse would probably occur only on the more heavily loaded side and with the pattern appropriate to the support conditions and the type of loading. His choice of symmetrical loading seemed the obvious one at this time; unsymmetrical loading could form a further study.

Mr. Dykes was pleased to learn that Mr. Percy had been encouraged to enquire further into the Fracture Line method. Theoretically, admitting the assumptions made, the fracture pattern could be represented by a series of straight lines, but the nature of the material being what it was, the formation of straight fracture lines on every occasion was more than could be reasonably expected. However he had been delighted and indeed proud of the patterns he had obtained and felt sure that he could represent, for instance, the typical fracture pattern illustrated for a Structure Type III, by straight lines without too much use of his imagination. It was true that in some circumstances the fracture lines might be curved, for example due to concentrated loads. There were, too, other complexities in the behaviour of these structures which might result in the appearance of curved fracture lines and if it were to any of these that Mr. Percy was referring and not merely to deviations in the main pattern, then much more detailed discussion of the problem than could be undertaken here would be required. Returning to the particular assumption of negligible elastic deformation required by Johansen, he commented that in his own tests of slabs measuring 3 ft. by 2 ft. with a thickness of 1 in. reinforced on the bottom, considerable elastic deformation occurred before the yield lines formed, but this did not appear to affect the fracture pattern.

Regarding the application of the method to full size structures he was confident that it could safely be used for the smaller roofs and those formed by completely precast units. He had in mind a case



Fracture pattern for Structure Type III.

where the two-slab roofs were of 24 ft. span and 8 or 9 ft. chord width, but he was reluctant to put forward any firm figures. He certainly could not recommend its use at the present stage of development for the design of large folded plate roofs.

Mr. Bennett had correctly outlined the mode of action of a shell structure and had made an astute observation regarding the traverse support conditions. Mr. Dykes fully appreciated the point and had tried to avoid using the term 'shell' to describe his structures. In prismatic shells and folded plate structures, bending moments played a much greater part and membrane forces a less important part than in curved shells and this was precisely the thought behind the current investigation. Current elastic design methods recognised the presence of membrane forces and attempted to calculate their magnitude, thereby implying that the plates were built-in at their ends, as Mr. Bennett had said. They also calculated bending moments, both longitudinal and transverse, on the contradictory assumption that a completely free support condition obtained at the traverses. This latter condition was deemed more appropriate in his case though the weakness was recognised, and further work in progress would investigate the effect of continuity at the traverses.

Both Mr. Davies and Mr. Ashdown had spoken on the same topic, that the Johansen fracture line theory seemed to give an ultimate load rather lower than the experimental one. He thought that this was generally accepted and his own tests on two-way slabs gave results in agreement with this characteristic. There seemed to be no definite answer to this but there was a belief, to which he subscribed, that an edge zone was formed around the slab, acting as a stiffening ring within which membrane tensions were set up. An article by Professor Ockleston in *The Structural Engineer*, Vol. XXXVI No. 6, June 1958, suggested that an arching action was present in a continuous two-way slab and could very well account for the 35 per cent difference mentioned by Mr. Ashdown. This idea, however, could not be applied in the same form to a simple two-way slab.

Replying to the Chairman, Mr. Dykes agreed that the boundary conditions here, as in every structure, without doubt represented an important aspect of the problem, and deserved a great deal of further study. He was satisfied that the idea he had presented would be amenable to modification to comply with any postulated conditions.

The cracks which had developed along the yield lines were tension cracks which eventually formed zones of plasticity in the material. If loading was continued, complete and actual collapse of the structure would occur as a consequence of the compression failure of the concrete on the other side of the slab.

A further point raised during the discussion concerned the proportions of the structures, which had been criticised. Folded plates will generally be thicker than a circular shell, at any rate for spans in excess of say 15 ft. A figure of 3 in. had been mentioned and this was perhaps on the low side; more often the thickness would be 5 or 6 in. for a structure of reasonable size. In this connection he quoted C. S. Whitney (U.S.A.), who had given 1/20 as a reasonable value for the thickness/slope length ratio of a slab element. The thickness of the test structures was 0·5 in. with a slope length of 12·3 in., giving a ratio of about 1/25. If the thickness were taken as the effective depth, this ratio would become 1/50. In this respect he thought the proportions were not unrealistic. As far as beam action was concerned, span/height ratios of approximately 9 for a single bay and 7 for multi-bay construction were common in the United States. The structures which had been discussed in the paper had span/height ratios of $4\frac{1}{2}$ and this admittedly was a long way from the value of 9, but in other test structures with a ratio of 8 the same sort of result had been obtained. No vertical cracking, as in a beam, had been observed.

In the construction of the test specimens, the end plates were formed $\frac{1}{2}$ in. proud of the shuttering and all the concreting was done by hand. No vibration had been carried out.

THE CHAIRMAN thanked Mr. Dykes very sincerely and cordially on behalf of the meeting for his replies to the questions raised and the observations made, and paid tribute to him again for his admirable handling of a difficult subject.

The President, 1960-61 — *Continued from page 297*

Consulting Engineer for the new Millbank Development near the Tate Gallery for Vickers Limited, which includes a 380 ft. high tower building and he was also the Consulting Engineer for the London Planetarium.

The Institution is to be congratulated on having as President for the coming Session one who has had such a wide experience and has gained so many distinctions in the field of structural engineering and its kindred professions. His enthusiasm and progressive outlook will be invaluable in the direction of the Institution's affairs. Already well known to members throughout the country, his year of office will undoubtedly bring him a host of new friends. Colonel Kirkland is assured of the good wishes of all members and those who will have the privilege of working under his guidance know that they may look forward with confidence to a year of continued progress for the Institution.

Institution Notices and Proceedings

PRESIDENTIAL ADDRESS

A General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 6th October, 1960, at 6 p.m., when Lt.-Colonel G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E., will be installed as President for the Session 1960-61 and will give the Presidential Address.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, 13th October, 1960.

Joint Meeting of the Institution of Structural Engineers and the British Section of the Société des Ingénieurs Civils de France at 6 p.m., when MM. Jean Bellier and Max Tourasse will read a paper entitled "Prestressed Concrete Pressure Vessels for Nuclear Reactors at Marcoule."

Advance copies of the paper may be obtained on application to the Secretary of the Institution.

Thursday, 27th October, 1960.

Ordinary General Meeting at 5.55 p.m. for the election of members. This meeting, which is open only to corporate members of the Institution, will be followed by an Ordinary Meeting at 6 p.m., when a paper entitled "Structural Considerations of a New Animal Feed Mill in Belfast, Northern Ireland" will be read by Mr. E. V. Finn, M.I.Struct.E., A.M.I.C.E. and Mr. K. F. Shadbolt, A.M.I.Struct.E.

Thursday, 10th November, 1960.

Ordinary Meeting at 6 p.m., when a paper entitled "The 16-in. E.R.W. Tube Plant, Shotton" will be given by Mr. W. T. Brooks, Mr. T. Burnett-Stuart (Graduate) and Mr. G. B. Godfrey, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E.

Thursday, 24th November, 1960.

Ordinary General Meeting for the election of members, 5 p.m.

Thursday, 8th December, 1960.

Ordinary Meeting at 6 p.m., when a paper on "The Structural Engineer in the Coal Industry" will be given by Mr. C. A. C. Davies, M.I.C.E.

Thursday, 15th December, 1960.

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a paper on "The London Traffic Problem" will be given by Mr. Donovan H. Lee, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., M.I.Mech.E. (Hon. Curator).

Members wishing to bring guests to the Joint Meeting and the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1961

The Examinations of the Institution will next be held in the United Kingdom and overseas on the 10th and 11th January, 1961 (Graduateship) and the 12th and 13th January, 1961 (Associate-Membership).

APPOINTMENT OF SECRETARY

Applications are invited for the position of Secretary of the Institution of Structural Engineers from men between the ages of 35 and 50. Applicants, who must be British, should possess a University degree, preferably with honours, or be a corporate member of a chartered engineering Institution, or have other suitable professional or service qualifications, and must have wide administrative experience.

Commencing salary will be £3,000 per annum; there is a contributory superannuation scheme.

The successful applicant will be required to take over from the retiring Secretary on the 1st October 1961.

Further particulars may be obtained on application to The Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1, the envelope to be marked "Appointment of Secretary."

The closing date for the submission of the detailed applications will be 30th November, 1960.

DRURY MEDAL AWARD—1961

The alternative subjects for this the eighth Competition, are a pipe-bridge and a petrol service station.

Graduates and Students of the Institution not over 27 years of age are invited to apply for full details to the Secretary, envelope to be marked in the top left-hand corner 'Drury Medal Award.'

The closing date for the competition is October 1st, 1961.

The general conditions of the competition are as follows:

(a) The competition shall be for Graduates and Students of not more than 27 years of age.

(b) The subjects of the competition will be designs of a structural character, that is to say, involving structural design rather than planning.

(c) The subjects of design and the conditions shall be prepared and issued biennially.

(d) A Jury shall be appointed to examine the work submitted and to interview candidates if found necessary.

(e) In order to ensure that the design submitted is the unaided work of the competitor, the drawings, calculations, etc., submitted shall be endorsed by the candidate: "I declare that the work I hereby submit is my own unaided work." The declaration shall be signed by the competitor, and be either countersigned by a corporate member, or be certified as made before a Justice of the Peace, or a Commissioner for Oaths.

SESSIONAL PROGRAMME

The Literature Committee have under consideration the selection of papers for inclusion in the Sessional Programme for 1961-2. Members who may wish to offer papers during the coming Session are invited to communicate with the Secretary.

ARTICLES FOR PUBLICATION

The Literature Committee would be glad to consider short articles of a descriptive character for publication in the Journal. The object of such articles would be to describe the difficulties encountered during con-

struction and the ways in which they were overcome ; and to set out details of structures successfully erected. Articles should be accompanied by good illustrations.

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :

The opening meeting of the Session will be held on Tuesday, 18th October, 1960, at the College of Science and Technology, Manchester, commencing at 7 p.m., when Professor W. Merchant, M.A., S.M., D.Sc., M.I.Struct.E., M.I.C.E., A.M.I.Mech.E., will give the Chairman's Address.

The meeting will be preceded by a Hot Pot Supper. A coach will run from Liverpool.

Thursday, 3rd November, 1960.

Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., (Vice-President), on " Foundations with Particular Reference to Piling." At Liverpool University in the New Civil Engineering Building.

Wednesday, 30th November, 1960.

" Some Unusual Aluminium Structures " by Mr. W. Hamilton, A.M.I.C.E., at the College of Building, Clarence Street, Liverpool.

Meetings will commence at 6.30 p.m. and will be preceded by light refreshments from 5.30 p.m.
Hon. Secretary : W. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs.

MIDLAND COUNTIES BRANCH

The Annual Dinner and Ladies' Evening will be held at the Botanical Gardens, Birmingham, on Saturday, 15th October, 1960.

A Meeting of the Branch will be held at the Byng Kendrick Suite, College of Advanced Technology, Gosta Green, Birmingham, 6, on Friday, 28th October, 1960, at 6.30 p.m., when the Chairman's Address will be given by Mr. G. Bottley, T.D., M.I.Struct.E. Tea will be served from 5.45 p.m.

Friday, 25th November, 1960.

Joint Meeting with the Birmingham and Five Counties Architectural Association. " The Architect and the Developer," by Mr. J. Seymour Harris, F.R.I.B.A., M.I.Struct.E., A.M.I.C.E.

Wednesday, 7th December, 1960.

Joint Meeting with the East Midlands Local Association of the Institution of Civil Engineers. " Concepts of Structural Safety," by Professor R. C. Coates, Ph.D., B.Sc., M.I.C.E., Professor of Civil Engineering, Nottingham University.

This meeting will be held at the East Midlands Electricity Board Showrooms, Irongate, Derby, at 6.15 p.m. Tea will be served from 5.45 p.m.
Hon. Secretary : S. M. Cooper, A.M.I.Struct.E., ' Applegarth,' Hyperion Road, Stourton, Nr. Stourbridge, Worcestershire.

GRADUATES' AND STUDENTS' SECTION

The first meeting of the Session will be held at the Engineering Centre, Stephenson Place, Birmingham, and will start at 6.30 p.m. on Friday, 4th November, 1960. A paper will be read by Mr. C. R. Blackwell, of Freeman, Fox and Partners, entitled " Design and Construction of the Volta Bridge." Tea will be served from 5.30 p.m.

Wednesday, 10th November, 1960.

" Precast Concrete Frame Construction " by Mr. Bryan E. Griffin, A.M.I.Struct.E., of Concrete Limited, at the Electricity Demonstration Hall, Irongate, Derby, at 6.15 p.m. preceded by tea from 5.30 p.m.

Friday, 2nd December, 1960.

Mr. J. W. Fortey, B.Sc., M.S., A.M.I.Struct.E., A.I.Mun.E., recently lecturer at the University of Texas, on " Structural Engineering in the U.S.A." This meeting will be of general interest, and will be open to wives and friends. To be held at the Engineering Centre, Stephenson Place, Birmingham, at 6.30 p.m. Tea will be served from 6. p.m.
Hon. Secretary : H. T. Dodd, Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwickshire.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :

Wednesday, 5th October, 1960.

At Newcastle. Address by the Chairman-elect, Mr. E. Ibbotson, M.I.Struct.E., at the Neville Hall, Newcastle, at 6.30 p.m. Buffet tea at 6 p.m.

Monday, 17th October, 1960.

At Middlesbrough. Installation of Branch Chairman by the Institution President Lt.-Col. G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E., Chairman's Address by Mr. E. Ibbotson, M.I.Struct.E., at the Cleveland Seientific and Technical Institution, at 6.30 p.m. Buffet tea at 6 p.m.

Tuesday, 1st November, 1960.

At Middlesbrough. " Lightweight Fire Protection and the Structural Engineer," by Mr. A. R. Mackay, A.M.I.Struct.E., at the Cleveland Scientific and Technical Institution. 6.30 p.m. with buffet tea at 6 p.m.

Wednesday, 2nd November, 1960.

At Newcastle. Mr. Mackay's paper will be repeated at the Neville Hall, Newcastle. 6.30 p.m. Buffet tea from 6 p.m.

Tuesday, 6th December, 1960.

At Newcastle. Joint Management Committee Meeting. Address by Mr. J. R. T. Gibson Jarvie of the United Dominions Trust. At the Neville Hall, Newcastle. 6.30 p.m. Buffet tea at 6 p.m.
Hon. Secretary : P. D. Newton, B.Sc., M.I.Struct.E., A.M.I.C.E., c/o S. Cussons & Partners, 112, Borough Road, Middlesbrough, Yorkshire.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :

Tuesday, 4th October, 1960.

The inaugural meeting of the Session. The Chairman's Address by Mr. W. A. Plester, M.I.Struct.E., at 6.30 p.m.

Tuesday, 1st November, 1960.

A paper " Design and Construction of a Factory " will be read by Mr. Harold Cunningham, B.Sc., A.M.I.C.E.

Tuesday, 6th December, 1960.

Joint Meeting with the N.I. Association of the Institution of Civil Engineers. Mr. F. R. Benson will read a paper on " Lift Slab Construction."

Meetings will be held in the Civil Engineering Department, David Keir Building, Queen's University, Belfast, at 6.30 p.m., unless otherwise notified. Tea will be served from 5.45 to 6.30 p.m.
Hon. Secretary : L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast, 5.

SCOTTISH BRANCH

The following meetings have been arranged :

Monday, 10*th October*, 1960.

The opening meeting of the Session at the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m. The Chairman's Address will be given by Mr. N. M. Brydon, M.B.E., B.Sc., M.I.Struct.E., M.I.C.E.

Tuesday, 11*th October*, 1960.

The Annual Dinner and Dance at the Grosvenor Restaurant, Gordon Street, Glasgow, at 6.30 p.m.

Tuesday, 8*th November*, 1960.

Joint Meeting with the Edinburgh Architectural Association, " The Relationship between Architect and Structural Engineer," by Mr. W. Underwood, F.R.I.B.A., at the Adelphi Hotel, Coburn Street, Edinburgh, at 6.15 p.m.

Tuesday, 15*th November*, 1960.

Joint Meeting with the West of Scotland Branch of the Institute of Welding. " Universal Beams—After Two Years " by Mr. A. W. Turner, at the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, 6.30 p.m.

Wednesday, 7*th December*, 1960.

Joint Meeting with the Edinburgh and East of Scotland Association of the Institution of Civil Engineers. " Zambesi Hydro Electric Development at Kariba—First Stage," by Mr. T. A. L. Paton, C.M.G., B.Sc., M.I.Struct.E., M.I.C.E., and Mr. C. I. Blackburn, at the North British Hotel, Edinburgh, at 6 p.m.

Tuesday, 13*th December*, 1960

" The Use of Timber as a Structural Medium," by Mr. J. H. Jaap, A.M.I.Struct.E., at The Institution of Engineers and Shipbuilders, Glasgow, 7 p.m.

Hon. Secretary : W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTH WESTERN COUNTIES SECTION

Hon. Secretary : C. J. Woodrow, J.P., ' Elstow,' Hartley Park Villas, Mannamead, Plymouth, Devon.

SOUTHERN BRANCH

The Inaugural meeting of the Branch will be held at Southampton on Friday, 4th November, 1960. The Chairman of the new Branch is Mr. J. J. Leeming, B.Sc., M.I.Struct.E., M.I.C.E.

Hon. Secretary : Mr. A. P. K. Tate, B.Sc., A.M.I.Struct.E., Department of Civil Engineering, The University, Southampton.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :

Wednesday, 2*nd November*, 1960.

The Chairman's Address will be given by Mr. D. Manolopoulos, M.I.Struct.E., at the Mackworth Hotel, High Street, Swansea at 6.30 p.m.

Thursday, 10*th November*, 1960.

Joint Meeting with the Institution of Civil Engineers. " Engineering Education in the U.S.A." by Professor B. Neal, M.A., Ph.D., A.M.I.C.E. at the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m.

Tuesday, 6*th December*, 1960.

Joint Meeting with the South Wales Institute of Architects, Western Branch. ' Brazil, Venezuela and Mexico' by Mr. Alex J. Gordon, Dip.Arch., A.R.I.B.A. at the Mackworth Hotel, Swansea, at 6. 30 p.m.

Hon. Secretary : W. D. Hollyman, A.M.I.Struct.E., 41, Greenfield Avenue, Dinas Powis, Glamorganshire.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :

Friday, 7*th October*, 1960.

The Chairman's Address by Mr. Clifford E. Saunders, M.I.Struct.E., A.M.Inst.W. Visit by the President and Secretary. Presentation of the Branch Prize and Headquarters awards to Derek Bond, A.M.I.Struct.E., Presentation of the Francis Memorial Prizes.

Thursday, 17*th November*, 1960.

Mr. F. C. Greenfield, A.Am.Soc.C.E., A.M.I.C.E., A.M.I.W.E. will present a paper entitled " The Trend in Engineering Works of Steel, Reinforced and Prestressed Concrete—some Continental Impressions."

Friday, 2*nd December*, 1960.

" The Characteristics of Friction Grip Bolt Joints," by Mr. M. S. G. Cullimore, B.Sc., Ph.D.

All meetings will be held in the small lecture theatre of the University Engineering Laboratories, University Walk, Bristol, 8, at 6 p.m. preceded by tea at 5.30 p.m.

Hon. Secretary : A. C. Hughes, M.Eng., A.M.I.Struct.E., A.M.I.C.E., 21, Great Brockeridge, Bristol, 9.

THE YORKSHIRE BRANCH

The following meetings have been arranged :

Wednesday, 19*th October*, 1960.

At Leeds. Dr. W. Eastwood, B.Eng.(Hons.), M.I.Struct.E., A.M.I.C.E., will give the Chairman's Address, at the Griffin Hotel, Leeds. Preceded by a buffet tea at 6.15 p.m.

Thursday, 20*th October*, 1960.

At Sheffield. The Chairman's Address will be repeated.

Wednesday, 16*th November*, 1960.

At Leeds. Joint Meeting with the West Yorkshire Society of Architects and the Yorkshire and Lincolnshire Branch of the Institution of Highway Engineers, ' Multi-Storey Car Parks,' by Mr. E. N. Underwood, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (Vice-President).

Wednesday, 7*th December*, 1960.

At Sheffield. " Recent Prestressed Concrete Work in North America and the Commonwealth " by Mr. Donovan H. Lee, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., M.I.Mech.E. (Hon. Curator).

Meetings at Leeds will be held at the Metropole Hotel, King Street, unless otherwise stated, and meetings at Sheffield at the Royal Victoria Hotel. Meetings will commence at 6.30 p.m., preceded by a buffet tea at 6.15 p.m.

Hon. Secretary : W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorkshire.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : E. B. Kretzschmar, A.M.I.Struct.E., P.O. Box 3306, Johannesburg, South Africa.

Natal Section Hon. Secretary : J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary : R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Hon. Secretary: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Hon. Secretary: W. N. Cursiter, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Redpath Brown & Co. Ltd., P.O. Box 648, Singapore.

NIGERIAN SECTION

Hon. Secretary: A. Brimer, A.M.I.Struct.E., Brimer, Andrews & Nachshen, Private Bag Mail 2295, Lagos, Nigeria.

SOME RECENT PUBLICATIONS OF INTEREST TO STRUCTURAL ENGINEERS

British Standards Institution—

B.S. Code of Practice: Traffic-bearing structures—Pavings. 30s. CP 2006 (1960).

Design and construction of reinforced and prestressed concrete structures for the storage of water and other aqueous liquids. CP 2007, 1960. 12s. 6d.

Amendment Slip to B.S. 706: 1936. Sandstone kerbs, channels, quadrants and setts. Amendment No. 1, PD 3670.

Amendment No. 3 *to B.S.* 785: 1938. Rolled steel bars and hard drawn steel wire for concrete reinforcement. PD 3676.

Amendment Slip to B.S. 2595: 1955. Descriptions and methods of measuring sizes and outputs of crushing machinery (when used for producing aggregates for constructional work.). PD 3682.

Amendment No. 3 *to B.S.* 2762: 1956. Notch ductile steel for general structural purposes. PD 3664.

Amendment Slip to B.S. 3139: 1959. High strength friction grip bolts for structural engineering. PD 3657.

CP 143—*Sheet Roof Coverings—Part* 3: 1960. Lead Roof Coverings. 7s. 6d.

B.S. 434: 1960 (*Revised Standard*). Bitumen Road Emulsion (anionic). 7s. 6d.

CP 143—*Sheet Roof Coverings—Part* 4: 1960. Copper Roof Coverings. 7s. 6d.

B.S. 938: 1955 (*Reprint*). General requirements for the metal-arc welding of weldable structural tubes. 4s. 6d.

B.S. 1549, Part 1: 1949 (*Reprint*). Methods for sheet metal pattern development. 3s. 6d.

*B.S. A*25: 1950 (*Reprint*). High tensile steel hexagonal-headed bolts. 3s.

B.S. 1728: 1960. Methods for the analysis of aluminium and aluminium alloys. *Part II*: Method for the determination of silicon (perchloric acid method). 3s.

CP 2006: 1960. Traffic bearing structures. Pavings 30s.

B.S. 706: 1936 (*Reviewed and confirmed*). Sandstone kerbs, channels, quadrants and setts (with published amendment).

B.S. 1984: 1953. Single-sized gravel aggregates for roads (with published amendment). (*Reviewed and confirmed*).

B.S. 2595: 1955. Descriptions and methods of measuring sizes and outputs of crushing machinery (when used for producing aggregates for constructional work) with published amendment. (*Reviewed and confirmed*).

Amendment Slip to B.S. 12: 1958. Portland Cement (ordinary and rapid-hardening). PD 3729.

Amendment Slip to B.S. 215: Part 1: 1956. Aluminium conductors for overhead power transmission purposes. PD 3751, Part 2: 1956 Steel-cored aluminium conductors for overhead power transmission purposes.PD 3752.

Amendment No. 5 *to B.S.* 825: 1949. Mild Steel Shackles.

Amendment Slip to B.S. 1370: 1958. Low heat Portland cement. PD 3734.

*ISO/R*129: *Engineering Drawing, Dimensioning.* (Obtainable from B.S.I. Sales Branch, 2, Park Street, London, W.1.) 11s. 6d.

B.S. 1121: *Part* 41: 1960. Methods for the analysis of iron and steel. Lead in carbon and low alloy steel. 3s.

Amendment Slips for B.S. 1470 to 1477: Wrought aluminium and aluminium alloys.

1470: 1955. Sheet and Strip. Amendment No. 2. PD 3778.

1471: 1955. Drawn Tube. Amendment No. 2. PD 3774.

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(184) *Atomic Energy Authority Act,* 1954. Balance Sheet of the United Kingdom Atomic Energy Authority as at March 31, 1959, together with the Report of the Comptroller and Auditor General thereon. 1s. 5d.

(185) *Historic Buildings Council for England.* Seventh Annual Report, 1959. 1s. 5d.

(187) *Historic Buildings Council for Wales.* Seventh Annual Report, 1959. Cyngor Adeiladau Hanesyddol Cymru. 11d.

(203) *Hydro-Electric Development* (*Scotland*) *Act* 1943. Statement of Guarantee given by the Treasury on March 31, 1960, in pursuance of Sect. 14 (1) of the Act, as amended by Sect. 47 (7) of the Electricity Act, 1947, as amended by the Hydro-Electric Development (Scotland) Act 1952, on Loans proposed to be raised by the North of Scotland Hydro-Electric Board. 5d.

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Standing Committee Debates. Official Report. Daily Parts (unrevised). Each 11d.

Corporate Bodies' Contracts. Standing Committee C. May 4, 1960.

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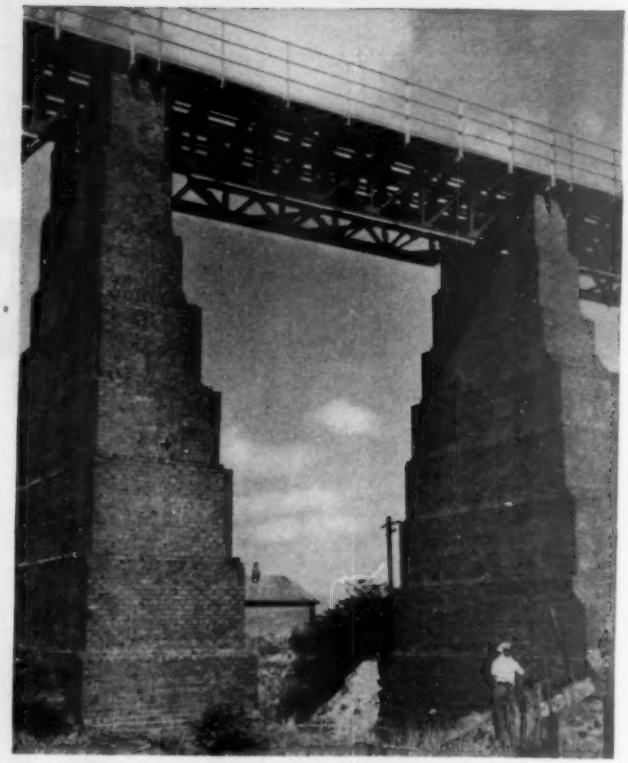

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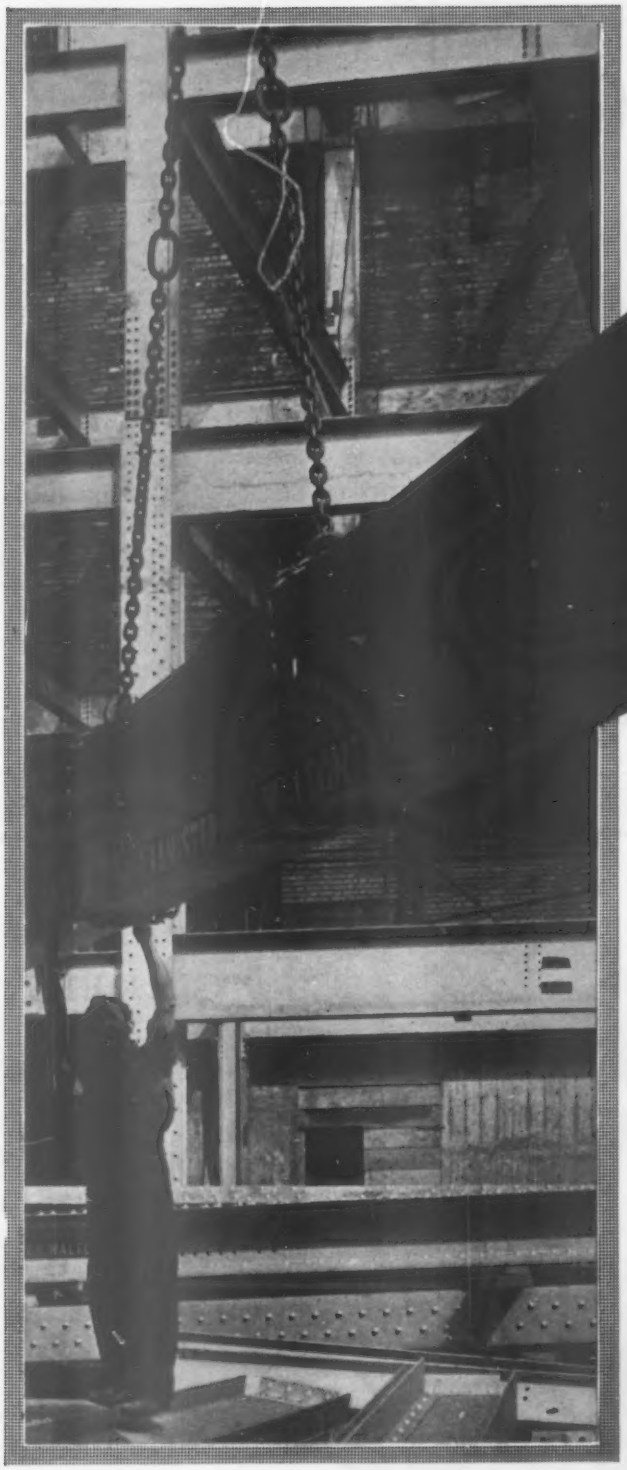

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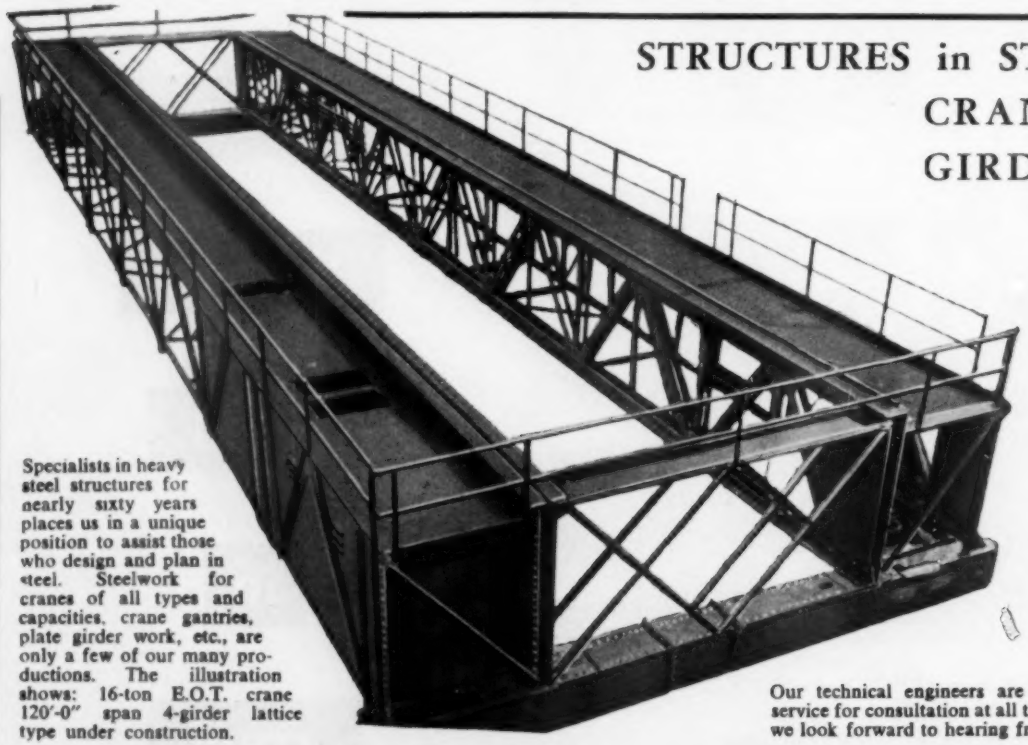

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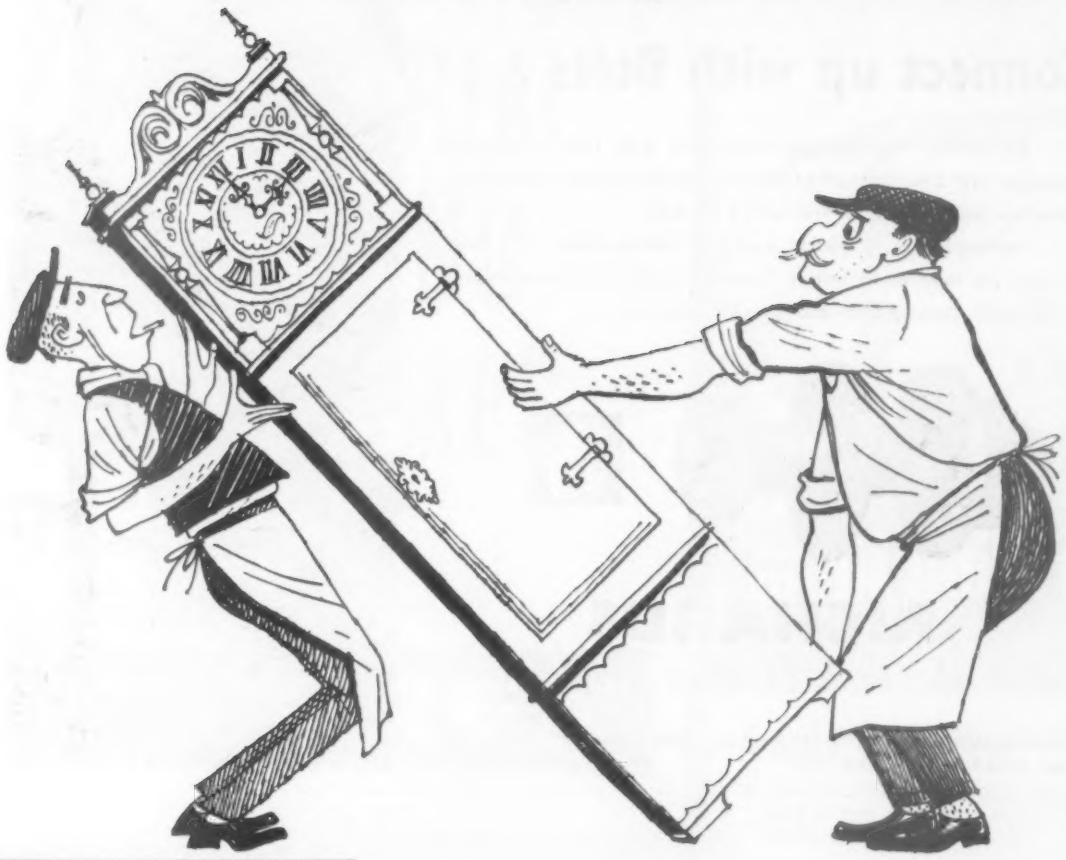

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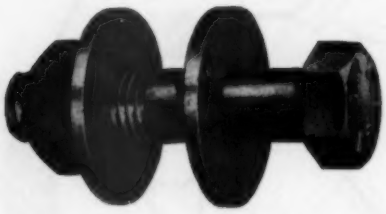

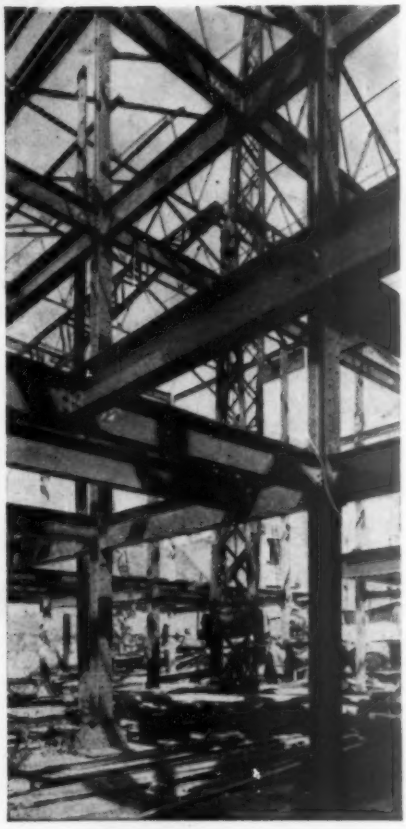

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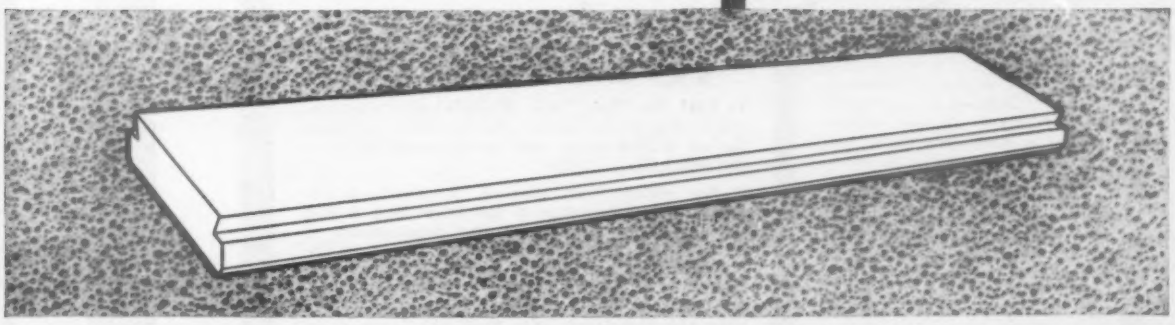

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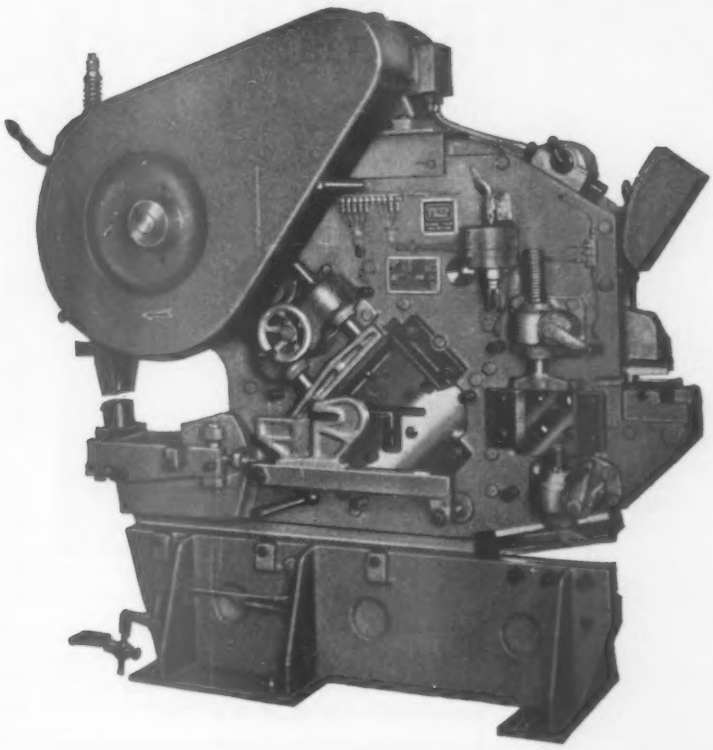

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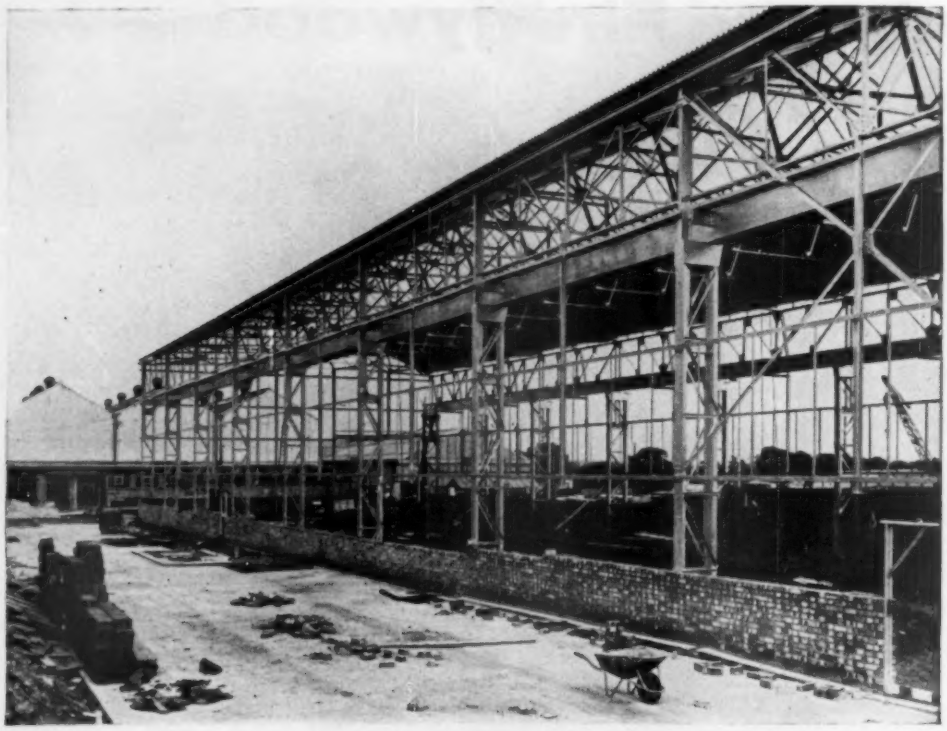

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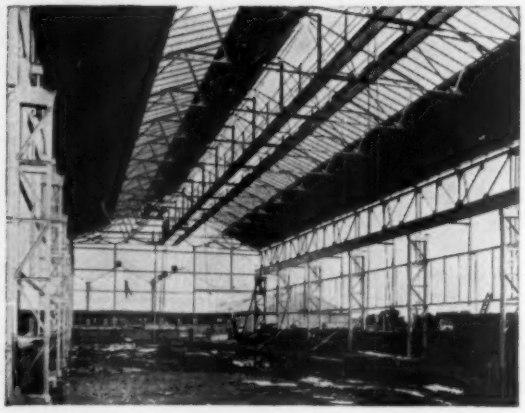

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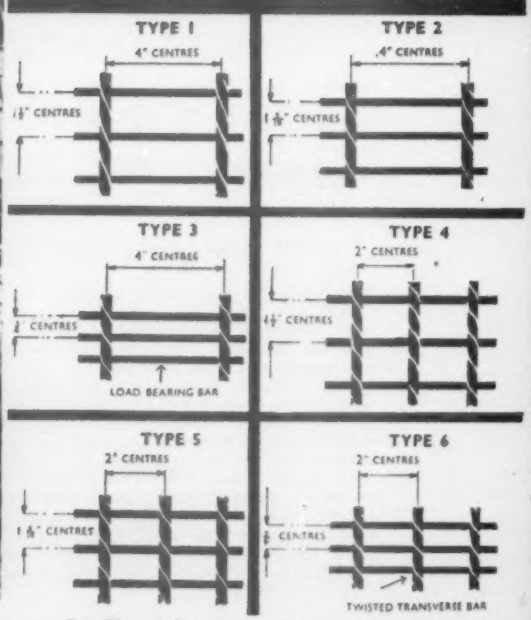

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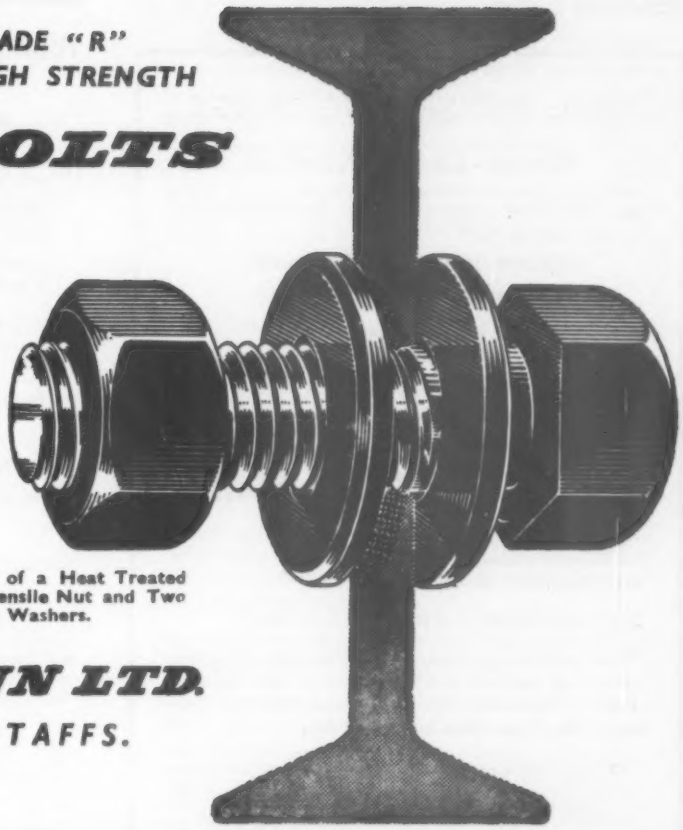
They give a pre-determined clamping effect and are not loosened by vibration.

Assembly of pre-fabricated structures on site, is speeded up.

No heavy riveting equipment is required and the absence of noise enables assembly to be done at night.



Assembly consisting of a Heat Treated Bolt and Medium Tensile Nut and Two hardened Washers.

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TURRIFF CONSTRUCTION CORPORATION LTD.

require several

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to cover expansion commitments in the structural and architectural departments of their Development Group. Minimum of three years experience necessary. Industrial training an advantage.

These appointments will be progressive and permanent carrying commencing salaries up to £1,000 p.a. with bonus and Pension Schemes applicable.

Excellent opportunity for specialistic experience and advancement.

Write for appointment,

Group Personnel Manager,
Turriff Building,
Great West Road, Brentford, Middx.

TURRIFF CONSTRUCTION CORPORATION LTD.

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is to appoint several

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each to cover a number of contracts co-ordinating the interests and efforts of Clients, design and construction teams. Must have comprehensive site and office experience. The wide interests of this Development Group will call for men of drive and initiative with either civil, structural, mechanical or architectural background.

Attractive salaries, bonus and Pension Scheme. Suitable transport provided.

Write full particulars to :

D. H. DUCKWORTH,
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Turriff Building,
Great West Road, Brentford, Middx.

SITUATIONS VACANT—continued

CIVIL ENGINEERING DESIGN

Simon-Carves Limited

have additional vacancies on the Civil Engineering Design staff of their

LONDON OFFICE IN BOND STREET

CIVIL ENGINEERING DESIGNERS

(a) STRUCTURAL

Applicants should have had previous experience of reinforced concrete structural design and detail. (Ref. UW. 91).

(b) GENERAL CIVIL WORKS

Applicants should have had experience in the design of effluent treatment systems, drainage, roads, and other services. (Ref. UW. 92)

Applicants should be of H.N.C. or equivalent.

These posts are permanent and pensionable. Starting salaries in accordance with experience and qualifications. Good prospects and working conditions, five day week, three weeks' annual holiday.

Send brief relevant details, quoting appropriate reference number to: Staff & Training Division, SIMON-CARVES LTD., Cheadle Heath, Stockport, Cheshire.

DESIGNER / DETAILERS

REINFORCED CONCRETE

THE MODULAR CONCRETE CO. LTD

Have vacancies in their Engineering Department for Designer/Detailers on interesting reinforced concrete work, both insitu and precast. Previous experience with specialist firm in this field desirable, but not essential. Working conditions are first-class. Basic salary up to £1,100 according to experience. Profit sharing bonus scheme after a probationary period. Superannuation Scheme. Subsidized canteen. Five day week.

Positions are progressive and offer excellent experience to energetic men interested in developing up-to-date techniques of design and construction.

Please apply in writing, stating age, training, experience and present salary to :

Personnel Manager,

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Norbury, S.W.16.

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DEVELOPMENT GROUP

A Midlands division of this London development group is to be etsablished permanently at Warwick immediately, and appointments will be offered to the following :—

ONE SENIOR ARCHITECT
TWO ARCHITECTURAL DRAUGHTSMEN
ONE SENIOR STRUCTURAL DESIGNER
TWO STRUCTURAL DRAUGHTSMEN

This is an excellent opportunity for men of ability and initiative to establish themselves advantageously in an expanding organisation.

Attractive salaries, bonus and pension.

Write or call:— Group Personnel Manager,
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2 in.	SEMI DISPLAY	£6	0s. 0d.
3 in.	"	£8	10s. 0d.
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CONSULTING ENGINEERS

have vacancies for Engineers and Draughtsmen for the design and detailing of a variety of building structures. The structures, large and small, mostly have strong architectural interest with the scope for intelligent structural design incorporating the use of concrete, brick, steel and timber with the emphasis on reinforced concrete.

Jobs are available in the following grades :—

(1) Senior Assistant Engineers. Preferably with good University Degree and professional qualifications and with at least seven years experience in reinforced concrete work. Age limits 30 to 40. Basic commencing salary £1,050 to £1,300.

(2) Experienced draughtsmen with five to ten years drawing office experience in reinforced concrete and preferably with some knowledge in, and aptitude for, design. Help will be given to those studying for professional examinations with opportunities (as they occur) to gain the necessary site experience. Age between 25 and 35. Approximate basic salary range £750 to £1,100.

In addition to the basic salary, generous annual bonuses are paid which reward efficiency and hard work. Luncheon vouchers, car allowance and travelling expenses between home and office are also paid. The working conditions are pleasant in new modern offices in Hammersmith. Five day week with good holidays.

Please write or telephone for interview to :—

CHARLES WEISS & PARTNERS,
123 KING STREET
HAMMERSMITH, W.6.
RIVerside 7843

Travelling (and hotel if necessary) expenses incurred for attending interview will be refunded.

HELICAL BAR & ENGINEERING CO. LTD.

have vacancies in their design office for experienced

ENGINEERS DETAILERS & JUNIOR DRAUGHTSMEN

The heavy demand for Helibond cold-worked reinforcement is creating wonderful opportunities for men with initiative, energy and drive.

Salaries offered

Engineers—£1,300 upwards
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University Graduates will be considered for some of these positions.

Five day week operating—L.V's. This year holiday arrangements honoured.

Apply in writing giving full details of age and experience to :—

HELICAL BAR & ENGINEERING CO. LTD.
82, Victoria Street, London, W.1.

LESLIE Turner & Partners, Consulting Engineers, have vacancies for Assistant Engineers experienced in reinforced concrete work. Applicants of any grade will be considered but employees of graduate status and earning not less than £700 per annum are urgently needed. Apply in writing to, 6, Grove Road, Sutton, Surrey.

OVE ARUP & PARTNERS

have vacancies in their

LONDON, MANCHESTER and EDINBURGH offices for Engineers, Designer Detailers and Detailers experienced in Reinforced Concrete, for work on a variety of interesting projects.
Interviews will be arranged at the appropriate office as convenient. Applications giving brief details of previous experience, qualifications, etc., should be sent to the address below.
Salaries offered will be according to ability and experience, and additional benefits include Luncheon Vouchers, Bonus and Voluntary Superannuation Scheme.

Ove Arup & Partners, 13, Fitzroy Street, London, W.1.

REEMA CONSTRUCTION LTD.

require urgently a Qualified Engineer and a Designer/Detailer for the Structural Drawing Office.

The work will consist of design on a variety of buildings including Multi-Storey Flats, involving the extensive use of precast concrete. A knowledge of reinforced concrete is essential.

Please write to Chief Engineer, Reema Construction Ltd., Milford Manor, Salisbury.

Reinforced Concrete Designers/Contractors, established 60 years, require Designer/Detailers for interesting work in modern Manchester Office, to work in a small team under qualified Engineers. Good salary and scope for advancement, five day week, pension scheme.

Apply Fram Reinforced Concrete Company Limited,
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Manchester, 13.
Telephone : Ardwick 4521.

SALES DIRECTOR REQUIRED

(London Structural Engineers)

Wanted by old established London Company, man of outstanding organising ability with thorough knowledge of the structural trade.

Applicants required to have had long experience in dealing with Architects and Consultants.

Salary commensurate with qualifications but most liberal terms offered to first class successful applicant. Apply with fullest particulars of experience in strictest confidence direct to The Chairman, if desired under nom de plume. Write to Box 9096, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

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required for Thermalite Ytong Limited, at their Central Offices, Nr. Birmingham. The appointment will be as deputy to the Chief Technical Officer and duties will include recommendations on forms of construction to architects and contractors, etc., with the use of the Company's materials.

Applicants should be either qualified architects, structural engineers or civil engineers. Age 24 to 32 years.

The position is in an already large but rapidly expanding Company, offering interesting work and the appointment is progressive and pensionable. The commencing salary will be in accordance with qualifications and experience.

Apply in writing stating age, qualifications, etc., to,

Personnel Manager (T.O.2) John Laing & Son (Holdings) Limited, Page Street, London, N.W.7.

REINFORCED Concrete detailer.—Orlit Limited have vacancy on reinforced concrete work both insitu and precast. Previous experience in this specialized field essential. Position is progressive and offers excellent scope for advancement. Apply in writing to, Contracts Director, Orlit Ltd., Vicarage Road, Egham, Surrey.

REINFORCED concrete detailers required by Leonard & Grant in their South London drawing offices. Only top men should apply. High salaries. Interview by appointment. Phone TATe Gallery 8496.

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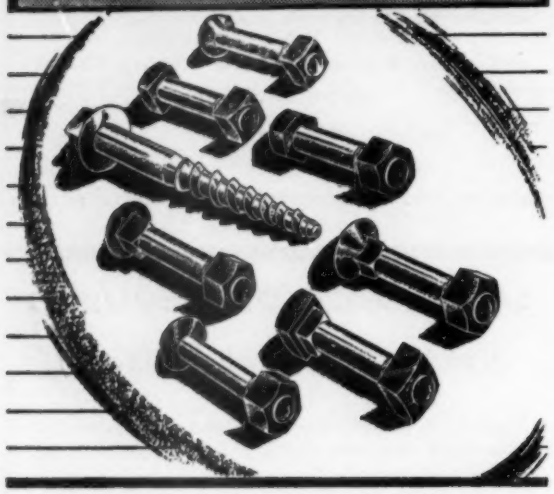

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SITUATIONS VACANT—continued

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Design and Supervisory Staff

One of Britains largest steelworks now has vacancies in its Development Department for:—

DESIGN ENGINEERS

with A.M.I.C.E., or A.M.I.Struct.E., or who hold equivalent qualifications. Experience in heavy industry structures would be an advantage.

DESIGN ASSISTANTS

who are registered graduates of the Institute of Structural Engineers, or hold H.N.C. with endorsements in Civil Engineering subjects. Experience in heavy industrial structures would be a decided asset.

CLERK OF WORKS

Structural Steelwork Clerks of Work who are required to inspect and supervise structural steelwork erection contracts. The successful candidate must have had experience in heavy industrial mill building type of construction, and preferably some previous experience as structural steelwork Inspector on the maintenance of structures applicable to the Iron and Steel Industry.

Please apply with details of experience, qualifications and age to:—
Box No. 9107, Structural Engineer, 43a, Streatham Hill, S.W.2.

STRUCTURAL ENGINEER

EXCEPTIONAL OPPORTUNITY

Consulting Civil Engineers, Westminster, require a fully qualified structural engineer with wide experience for position as chief designer in reinforced and prestressed concrete and structural steelwork, including bridges and buildings. Knowledge of modern bridge design essential. Prospects of partnership interest in firm.

Write in confidence, stating age and full details of experience and qualifications. Box 9095, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

STAFF required for Consulting Structural Engineer's office. Senior and Junior engineers and detailers, experienced in R.C. framed structures. Five day week; Luncheon vouchers. Apply in writing to, Felix J. Samuely & Partners, 231/233, Gower Street, N.W.1, giving age, experience and salary required.

STRUCTURAL draughtsman required. Good prospects; New department. Apply Sid Cubbage (Engineers) Ltd., Coronation Road, High Wycombe. (3166).

STRUCTURAL Estimator required to take charge of estimating department of rapidly expanding firm specialising in steel portal frame buildings. Experience in costing steelwork fabrication, erection and sub-contract items, and knowledge of building construction essential. Contracts with suppliers an advantage. Age 30 to 50. Good salary plus share in profits. Pension scheme with Free Life Insurance. Apply in writing to, Conder Engineering Co. Ltd., Winchester, Hants.

STRUCTURAL Steelwork designer with a good knowledge of light structures as well as conventional steel design required by Consulting Engineers. Five day week, pension scheme, holiday arrangements honoured. Apply stating full details of experience and salary required to, Alan Marshall & Partners, Federal House, Down Place, W.6.

STEELWORK Designer. Fully qualified structural designer required. Must be able to negotiate and supervise contracts and maintain close contact with important clients. Pleasant personality and plenty of drive essential. Age 26 to 40. The position offers scope for advancement in a Company with an exceptional record of expansion. Good salary; Excellent profit sharing scheme; Pension scheme and Free Life Insurance. The vacancy is created by promotion within the Company. Apply Conder Engineering Co. Ltd., Winchester, Hants.

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CHARTERED Structural Engineer—eight years North American experience bridges and heavy structures, interested in partnership with Consultant—Southern or Western England. Capital available. Box 9108, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

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HEAVY Portable Hydraulic Riveting Machine for sale by Fielding & Platt. For suspension from crane. Cap. 1″ rivets. Depth of gap 24½″. Height of gap without bolsters 15″. Full movement in hanger. Photo F. J. Edwards Limited, 359, Euston Road, London, N.W.1.

PELS Angle and Tee Cropper for sale. Double ended on turntable base. Crops angles 7″ × 7″ × ⅞″ or 6″ × 6″ × ¾″ or 6½″ × 6½″ × ¾″ at right angles or on the bevel. Motor drive 400/3/50. All steel construction. F. J. Edwards Ltd., 359, Euston Road, London, N.W.1, or 41, Water Street, Birmingham, 3.

PELS Type EFG 20. Section Cropping Machine for sale. Steel plate frame. Crops rounds up to 2⅜″, squares 2″. Flats 3″ × 1¼″ or 6″ × 15/16″, angles and tees at right angles 6″ × 6″ × ⅝″, angles and tees on mitre 4″ × 4″ × ½″. Arranged motor drive 400/3/50. Weight about 50 cwts. F. J. Edwards Ltd., 359, Euston Road, London, N.W.1, or, 41, Water Street, Birmingham 3.

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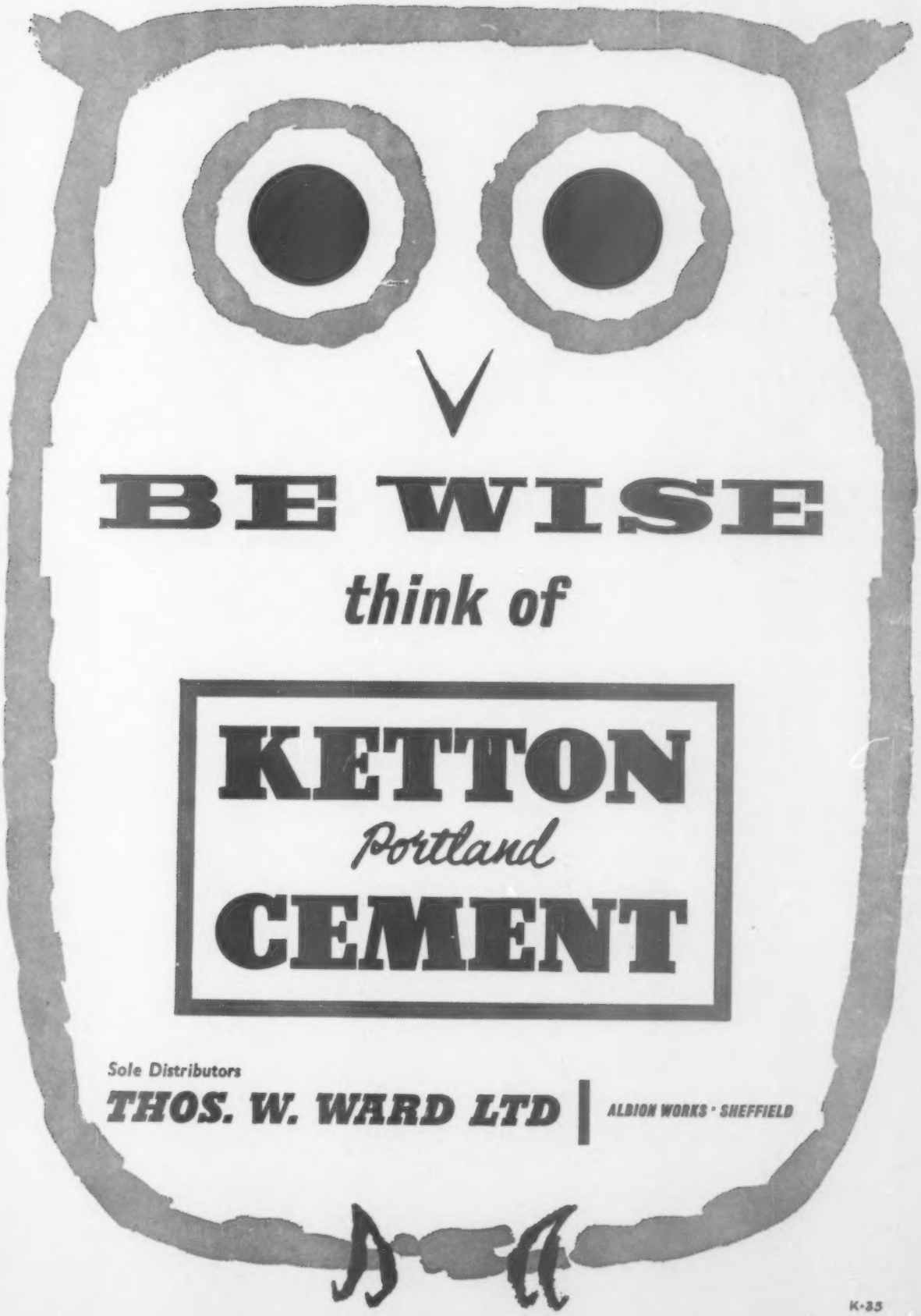
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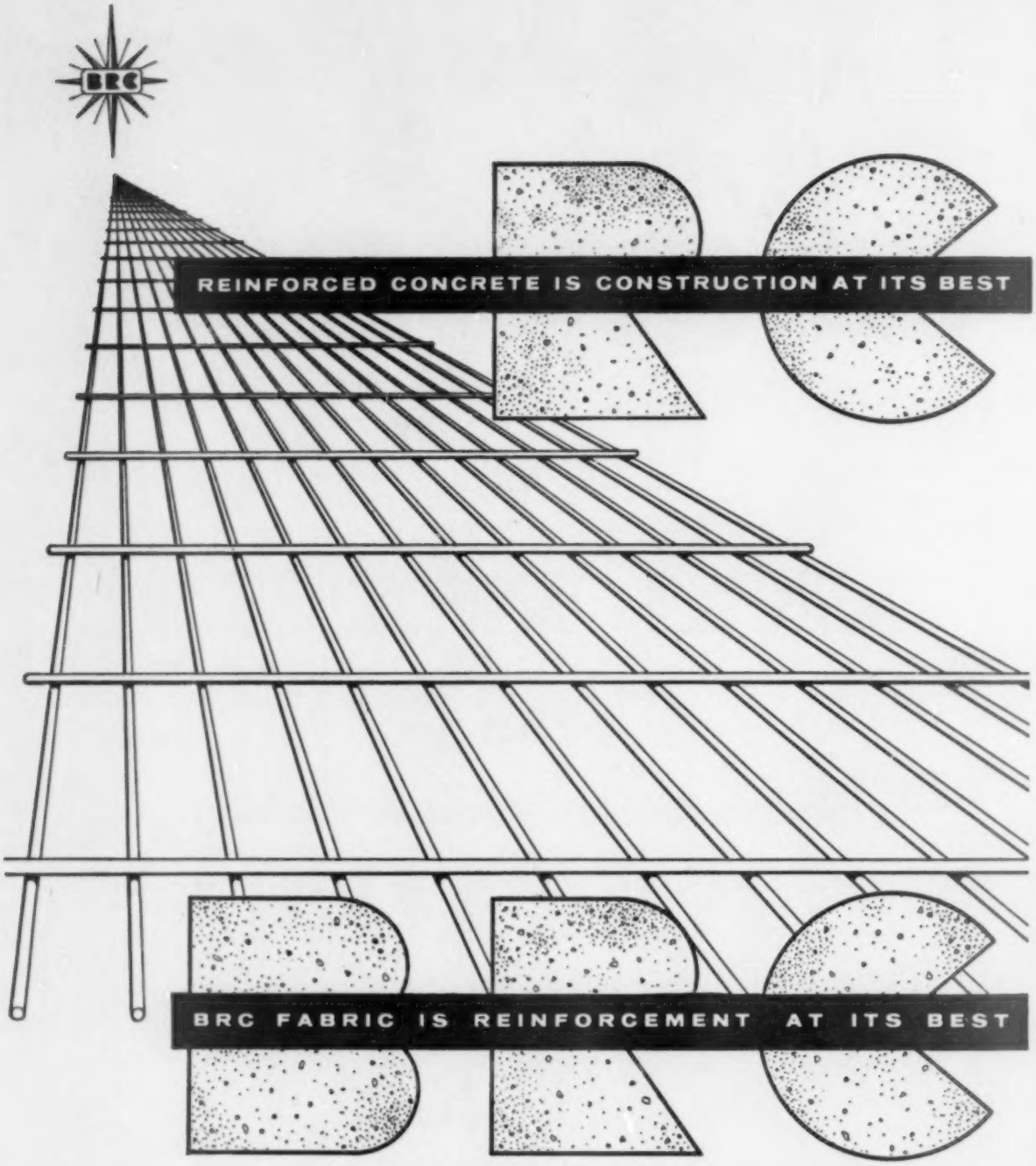

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